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Appendixes K, L, M, and N

for

Preliminary Design Study for a Dynamic Soil Testing Laboratory

Massachusetts Institute of Technology
Department of Civil and Sanitary Engineering
Soil Engineering Division

August 1961

PRELIMINARY DESIGN STUDY FOR A DYNAMIC SOIL TESTING LABORATORY

Appendixes K, L, M, and N

Massachusetts Institute of Technology
Department of Civil and Sanitary Engineering
Soil Engineering Division

August 1961

Research Directorate
AIR FORCE SPECIAL WEAPONS CENTER
Air Force Systems Command
Kirtland Air Force Base
New Mexico

Approved:

Donald I. Prickett

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Colonel USAF
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Project 1080
Task 10801
Contract AF29(601)-1947

ABSTRACT

Four laboratory experiments performed on small samples of soils are discussed. The experiments were designed to furnish certain information on soil behavior needed for the design of a dynamic soil testing laboratory. The four experiments studied friction angles of sands at low confining pressures, side friction in the consolidation test, failure conditions in hollow soil cylinders, and failures in tubes surrounded by soil. The apparatus and procedures used in each case are described, and typical data and results are presented.

PUBLICATION REVIEW

This report has been reviewed and is approved.

John J. Dishuck

JOHN J. DISHUCK
Colonel USAF
Deputy Chief of Staff for Operations

PREFACE

The work reported in Appendices K, L, M, and N included under this cover was carried out under Contract No. AF 29 (601) - 1947, "Preliminary Design Study for a Dynamic Soil Testing Laboratory", and constitutes part of the final report in fulfillment of that contract.

All four appendices describe small-scale tests concerned with soil reactions or soil-structure interactions which have bearing on the design of a Dynamic Soil Testing Laboratory. During the first year of the contract, however, the testing proved to be going so well that the contract was extended beyond the interests of the design of the laboratory to more elaborate studies of soil-structure interaction.

Because the tests described herein may be of general interest to the Civil Engineering profession, these appendices are being published separately from the remainder of the report. The work was performed principally by Ulrich Luscher, Instructor, and Allen R. Philippe, Research Assistant, with the help of the staff of the Soil Engineering Division, Department of Civil and Sanitary Engineering. Professor Robert V. Whitman is the principal investigator under this contract. Professor T. W. Lambe is Head of the Soil Engineering Division.

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Appendix K

TESTS ON HOLLOW SOIL CYLINDERS

Appendix K

TESTS ON HOLLOW SOIL CYLINDERS

K.1 Background

During the past two years, the Massachusetts Institute of Technology, in connection with Contract No. AF 29(601)-1947, has conducted very basic small-scale tests as a means of studying fundamentals of soil-structure interaction. These tests involved hollow cylinders of soil on one hand, and flexible cylinders of plastic or metal, surrounded by a thin layer of soil, on the other hand. This appendix describes the work done on hollow soil cylinders.

Figure K.1 indicates roughly the type of test that was performed. A hollow cylinder of sand is encased on both the outside and the inside with a thin flexible membrane. To begin the test, the pressures within and without the cylinder are raised equally to a value above atmospheric pressure. Subsequently, either the outside pressure is increased or the inside pressure is decreased, until the cylinder collapses inward. During all this time the pore pressure in the soil is at atmospheric level. The loading arrangements are such that there is no axial strain in the hollow cylinder of soil.

In the first phase of this work, executed during the academic year 1959-1960, the equipment was developed and built, a number of tests were performed, and a theory for failure of hollow soil cylinders was developed. The results of this work were very encouraging but due to difficulties arising in the test procedure, many of the specific data were not very reliable.

In a subsequent second phase, during the academic year 1960-1961, the equipment and testing procedures were modified and improved. Better experimental results could then be obtained, and consequently more reliable

conclusions could be reached on the problem of correlation between experimental data and theoretically derived relationships.

This appendix will only briefly summarize the conclusions drawn from the first phase of the work, and otherwise will deal with the results of the second phase, since due to the changes in equipment and measuring techniques, the results of the two phases could hardly be correlated.

K.2 Theoretical strength law

K.2.1 Failure of soil element

An element of cohesionless soil with friction angle ϕ reaches a state of plastic equilibrium at a maximum ratio of the principal stresses,

$$\lambda = \left(\frac{\sigma_3}{\sigma_1}\right)_{max} = \frac{1-\sin\phi}{1+\sin\phi} \qquad \text{(Eqn. K.1)}$$

where σ_3 = minor principal stress = radial stress (see Figure K.1)

σ_1 = major principal stress $\doteq$ circumferential stress

K.2.2 Failure of soil ring

A λ - failure condition is assumed to hold for every element in the ring; i.e., if according to elastic stress conditions a λ - condition is first reached at the most critical point in the ring (which happens to be on the inside boundary), this element remains in plastic equilibrium and when the applied load is increased, adjacent elements reach a state of plastic equilibrium. Collapse of the ring is achieved when the ring reaches a λ - condition throughout.

The following derivation is a modification of calculations published by W.M. KIRKPATRICK (1953). Plastic equilibrium of a ring element between radii r_1 and r is considered (see Figure 1):

$$p_1 r_1 - \sigma_r r + \int_{r_1}^{r} \sigma_t \, dr = 0$$

After differentiation: $$-\frac{d\sigma_r}{d_r} r - \sigma_r + \sigma_t = 0$$

Substituting: $$\sigma_t = \frac{\sigma_r}{\lambda}$$

And rearranging: $$\frac{d\sigma_r}{\sigma_r} \frac{\lambda}{1-\lambda} = \frac{dr}{r}$$

After integration: $$\sigma_r = A\, r^{\frac{1-\lambda}{\lambda}}$$

Boundary condition: $$p_1 = A\, r_1^{\frac{1-\lambda}{\lambda}} \longrightarrow A = p_1 \frac{1}{r_1^{\frac{1-\lambda}{\lambda}}}$$

Substituted for A: $$\sigma_r = p_1 \left(\frac{r}{r_1}\right)^{\frac{1-\lambda}{\lambda}}$$

Specifically, at r_o: $$p_o = p_1 \left(\frac{r_o}{r_1}\right)^{\frac{1-\lambda}{\lambda}}$$

Final form: $$p_o/p_1 = \gamma = \left(\frac{r_o}{r_1}\right)^{\frac{1-\lambda}{\lambda}} \qquad \text{(Eqn. K.2)}$$

The ratio γ as expressed by this theory shows fast increase with increasing r_o/r_1, but it does not tend to infinity for any finite r_o/r_1. The character of this relationship is as would be expected for a cohesionless soil.

Another theory of ring strength, which assumed a λ - condition for average radial and tangential stresses in the ring, was investigated for its merits and rejected, because γ tended to infinity for finite r_o/r_1.

K.2.3 Numerical examples

K.2.3.1 Friction angle of Ottawa Sand

Results of recent tests at M.I.T. for the well-graded Ottawa Sand used in these tests are shown in Figure K.9. From that curve, and in agreement with results of tests on standard Ottawa Sand described in M.I.T. (1953), p. 104 and 105, typical friction angles

are obtained:

Dense state:	$\varphi = 38^{\circ}$	$\lambda = 0.238$
Loose state:	$\varphi = 32^{\circ}$	$\lambda = 0.308$

K.2.3.2 Prediction of ring strengths

Using equation K.2 and the above λ values for Ottawa Sand, the strength of sand rings was predicted in terms of the pressure ratio γ at failure.

TABLE K.1 PREDICTED RING STRENGTHS

Geometry of Soil Ring				γ for dense sand	γ for loose sand
r_o	r_i	$r_o - r_i$	r_o/r_i		
3/4"	1/2"	1/4"	1.5	3.66	2.49
1"	1/2"	1/2"	2.0	9.20	4.75
1 1/2"	1/2"	1"	3.0	33.7	10.6

These values point out the rapid strength increase with increasing ring thickness, and the extreme sensitivity of γ to changes in the soil density.

K.3 Testing technique

K.3.1 Equipment

A modified Norwegian Geotechnical Institute triaxial cell (with a two inch opening in the base plate) was used as the pressure vessel.

Main design specifications for the lucite base and cap assembly were: a) to hold the outside and the inside membrane in place; b) to allow the preparation of the sample, and c) to prevent any displacement of the top assembly, so that the soil approaches a state of plane strain. To satisfy (a), a system with conical lucite pieces and O-rings was devised to hold

the inner membrane. A central steel rod satisfied the requirements of specification (c). The testing device is shown in Figure K.2.

Before Phase Two was begun, the original equipment was somewhat modified, based on the experiences of Phase One. The most important change involved use of new membranes. In the tests of Phase One, a double thickness of commercial rubber prophylactics had been used for diameters of 1" and $1\frac{1}{2}$", while in the later tests, "homemade" rubber membranes were used. These membranes had the advantages of much higher resistance against mechanical damage and the possibilities of desired variations in size and thickness. They were produced at a standard thickness of 6/1000 of an inch, by a method described in HARVARD (1949).

K.3.2 Preparation of samples

Well-graded dry Ottawa Sand was used at void ratios between 0.48 and 0.55. The friction angles of this material as a function of the void ratio, were determined by triaxial tests on 2.8" samples, and the results can be seen in Figure 9.

During filling-in and compacting of the soil, both membranes were backed up by molds. When the surface of the soil had reached the desired level, the main top lucite piece was put in place, the membranes were connected to it, a full vacuum was applied to the soil, the molds were removed, and the lucite cover piece was screwed on. Then accurate initial sample dimensions were measured, and the sample was ready for testing.

K.3.3 Observations

The strength data from all tests in terms of $\gamma = p_o/p_i$ at failure were obtained. If these data alone were desired, the procedure was to apply increasing equal gas pressures to the cylinder cavity and outside chamber while reducing the soil vacuum to zero, then, with the soil under atmospheric pressure, to increase the outside pressure, while keeping the inside pressure constant, until failure took place. This

procedure was used only in a few tests of the First Phase.

In all other tests, in addition to strength data, deformations of the hollow soil cylinder were also observed. For this purpose, the total volume changes undergone by the inner and outer surfaces were measured as pressure was applied. Due to the elastic properties of the pressure cell, plastic tubing, and burette, the pressure of the volume to be measured had to be kept constant. Consequently, in tests with measurement of outside volume changes, the outside pressure was kept constant during the test and the inside pressure lowered. When inside volume changes were measured, inside pressures were kept constant and outside pressures were raised to failure.

In the Second Phase of the work, an attempt was also made to measure diameter changes or circumference changes directly by means of mechanical devices attached to the sample. One of these is shown in Figure 3a. Unfortunately, these devices could never be made to work properly, and additionally there was evidence that the presence of the device (having some rigidity) altered the failure mode and, thereby, probably the strength properties of the sample. Due to lack of time for further experimentation, this method therefore, had to be abandoned.

K.3.4 Failure

Failure occurred in all cases suddenly, with the sample caving in along a characteristic failure line which is straight in its central part and bent over near the ends. Some typical failure lines are shown in Figures K.3a and K.3b. Figure K.3b also shows a typical failed sample cross-section.

The nature of the failure phenomena corroborates the assumption that collapse is, in fact, caused by ring compression failure, which forms the basis of the theoretical considerations. It also verifies the tacit assumption that the longitudinal cylinder stresses are in the role

of intermediate principal stresses, at least over the central part of the sample.

K.4 Testing Phase One

K.4.1 Description

In this phase a number of tests were run on samples of $\frac{1}{4}$", $\frac{1}{2}$", and 1" cylinder wall thicknesses and 5" height. In a number of the tests, volume changes of the outside or the inside surfaces of the samples were observed.

It was attempted to compact all samples as closely as possible to the same density. Within each group, this could be achieved fairly well (strength results of tests on samples with odd densities were adjusted to correspond to the mean density of the group), but the mean calculated void ratios of the groups of tests were considerably varied, probably due to the different compaction characteristics of samples of different wall thicknesses, and possibly due to systematic errors in the determination of the void ratios.

A number of other difficulties became apparent and had to be overcome one by one. Most troublesome was the elimination of all possibilities of leakages.

K.4.2 Results

The significance of this first testing phase was twofold: first, it gave the opportunity for perfecting the equipment and testing techniques: and secondly, it provided preliminary results, which can be summarized as follows:

a) Cylinder geometry and soil density being equal, a plot of p_i versus p_o at failure resulted in a straight line. Such a result indicates that the strength of hollow soil

cylinders can be expressed as the ratio of outside to inside pressure at failure.

b) The strength ratio is a function of sample geometry and soil friction angle. Soil friction angles, necessary to explain the observed strength backcalculated by the theory developed in Section K.2, were of the right order of magnitude. The evidence was, however, insufficient to verify the theoretical relationship. It was thought at the time that end-effects due to the lateral supports and systematic errors in obtaining the void ratio distorted the results.

c) Crude values of the sample deformations, both on the outside and on the inside, could be obtained by volume change measurements. It was seen from these values that a significant decrease in inside diameter was necessary to mobilize the full strength of the soil cylinder.

K.5 Testing Phase Two

K.5.1 Description of tests

In this phase, tests were run on cylinders of two nominal wall thicknesses, $\frac{1}{4}$" and $\frac{1}{2}$", two different lengths, 5" and 3" ("long" and "short" samples); and at void ratios varying over a range of 0.49 to 0.53. In almost all tests either the inside or the outside volume change was observed. Additionally the rate of load application was varied over a certain range and roughly recorded. A listing of pertinent data of all tests in this phase is given in Table K.2.

K.5.2 Results

K.5.2.1 Main strength data

Taking advantage of the conclusion reached in Phase

TABLE K.2 LIST OF TESTS OF PHASE TWO

Test No.	Nominal Wall Thickness	Approximate Length	Void Ratio e	At Failure			Loading Time (last 50% of load)	Volume Change at Failure		Location of Vol. Change Discontinuity	
				Outside Pressure	Inside Pressure	$\gamma = p_o/p_i$		Outside	Inside	Upper Limit	Lower Limit
101			.537	63.2	14.2	4.45	$13\frac{1}{2}$		3.25	3.82	3.60
102			.528	45.0	10.2	4.41	$6\frac{1}{2}$	3.9		3.75	3.22
103			.508	74.3	14.3	5.20	20		3.35		
104	$\frac{1}{4}$"	5"	.515	50	10.6	4.70	7	3.1		3.57	3.12
105			.508	69	14	4.92	$14\frac{1}{2}$	4.2		3.84	3.45
106			.507	60	11.3	5.30	13	2.6		3.75	3.33
111			.495	68.75	12.75	5.40	8		2.8		
115	$\frac{1}{4}$"	3"	.550	60.0	14.0	4.28	12	3.7		3.62	3.33
112			.499	65.0	18.0	3.60	2				
201			.500	50	4.15	12.3	4				
202			.515	38	3.76	10.1	2				
203	$\frac{1}{2}$"	5"	.521	80	5.98	13.4	15		5.4		
204			.505	56.5	5.00	11.3	6	8.0			
205			.491	98	5.30	18.5	12		9.0	13.9	14.4
206			.497	70	4.17	16.8	9		7.1	11.8	12.1
211	$\frac{1}{2}$"	3"	.487	60	3.43	17.5	$10\frac{1}{2}$	3.8		14.0	14.5
212			.485	100	4.8	20.8	16		5.6		

One, namely, that for a given density and geometry, all data points lie on a straight line in a plot of inside versus outside pressure at failure, the ratio $\gamma = p_o/p_i$ of the pressures expresses the strength result of a test. For each cylinder wall thickness this ratio was plotted versus the initial void ratio of the sample. The plots are shown in Figures K.4 and K.6 for the two wall thicknesses, respectively, whereby both the data points from the "long" and from the "short" samples are shown on the same plot. An inspection of these plots shows:

a) For $\frac{1}{4}$" wall thickness, all points, with the exception of point 12, lie close to a single line, henceforth called the "main strength line".

b) For $\frac{1}{2}$" wall thickness, all points lie close to either one of two lines. The upper line is the main strength line, whereas the lower line is a line representing some type of premature failure.

From the common main strength line for "long" and "short" samples, it can be concluded that the strength of hollow soil cylinders is approximately independent of length, i.e., lateral supports have no effect on strength.

K.5.2.2 Volume change at failure

For each wall thickness, inside and outside volume changes were plotted against void ratio. These plots are shown in Figures 5 and 7. In plotting the data from the tests on "short" samples, it was found that by multiplying the volume changes by 5/3 (the ratio of the two sample lengths) the best possible correlation between the data for long and short samples was achieved. Corresponding points could then be reasonably well connected by straight lines in all cases, so that only one data point was significantly off (by more than 1 cm^3).

A rough examination shows that the outside volume changes increase with increasing void ratio, whereas the inside volume changes decrease.

The difference between outside and inside volume changes represents the volume decrease of the soil sample (positive at large void ratios; negative, i.e., dilatation, at small void ratios). This agrees qualitatively with the known fact that loose sands consolidate and dense sands expand under shear.

The good fitting-in of the adjusted "short" sample data in all cases indicates that the final volume changes of the cylinders are indeed proportional to the cylinder length, as was arbitrarily assumed earlier. This fact, together with the known phenomenon of equal strength of samples with different lengths, can only mean that the samples deform cylindrically over most of their length, which is also proved by many of the observed failure lines (see in Figure K.3).

K.5.2.3 Development of volume changes under loading

Some typical curves of volume change versus pressure ratio are shown in Figure K.8. It is interesting to note that many of the curves, especially those for moderately fast tests, show a distinctive discontinuity at somewhere between 60% and 80% of the final pressure ratio. Due to the fact that the pressures were applied in finite steps, only a possible range of the exact location of the break (in terms of γ) can be given. This range is shown on the strength plots, Figures K.4 and K.6, for all samples exhibiting the sudden break.

K.5.2.4 Time effects

Unfortunately the rate of loading in the experiment was not as well controlled as would have seemed desirable in retrospect. The reason for this omission was the belief that, as long as the load was applied relatively slowly, no significant effects of the loading rate would be encountered. Nevertheless, the time history of loading of all except the fastest tests was approximately recorded, and these records proved to be of great help in correlating and explaining many of the test results, which did indeed show a time effect. The following specific time

effects became evident from these efforts:

a) Strength results: Correlating strength data and loading rates, it is seen that all data points falling much below the main strength lines stem from tests with loading times of less than six minutes for the last 50% of the load, whereas all data points lying on the main strength lines are for loading times larger than six minutes.

b) The discontinuity of the volume change curve mentioned earlier was observed in all tests reaching the main strength line, except the slowest ones.

K.5.2.5 <u>Premature failure</u>

Figures K.4 and K.6 show that all strength data points falling much below the main strength line (i.e., points representing premature failure), and the ranges of sudden volume changes can approximately be connected by a single line in each plot. This common line suggests the following: apparently there is a critical stage in the development of the ring strength, at which readjustments of considerable magnitude have to be made in order for the sample to accommodate higher loads. Depending on the rate of loading, three types of behavior are possible at this stage: a) If the load is raised rapidly past this point (before the relatively slow adjustments could take place), the sample will fail prematurely; b) If the loading rate is moderate, the adjustments are liable to take place suddenly, as evidenced by the observed sudden large deformations, but collapse is avoided; and c) If the loading rate is slow, no recognizable discontinuity in the volume change curve will be obtained.

K.5.3 <u>Preliminary summary of results</u>

It seems useful at this time to summarize briefly the tentative conclusions which have been reached so far, i.e., before any attempt is made to correlate the results of the tests on cylinders of varying thicknesses:

a) For each wall thickness, the strength of reasonably slowly loaded samples is a unique function of the initial void ratio of the sample, independent of sample length.

b) For each wall thickness, specific volume changes at failure, i.e., volume changes per unit length of the cylinders, both inside and outside, are unique functions of the initial void ratio. The near occurrence of "premature failure" does not seem to affect volume change at "main failure", since, after the sudden large volume change, almost no further volume change takes place until failure is reached (e.g., Test 115, Figure K.8), whereas volume change curves of samples not experiencing the discontinuity get continuously steeper until failure occurs (e.g., Test 111, Figure K.8).

c) There is a critical load below the main failure load, at which rapidly loaded samples fail prematurely, and samples loaded at an intermediate rate experience a sudden large deformation without failure. This critical load is also a unique function of the void ratio for each wall thickness.

K.6 Evaluation and discussion of results

K.6.1 Main strength data

Evaluation of the strength data is done by backcalculating the friction angles of the soil from the test results, using the theoretical strength relationship derived in Section K.2. This procedure is justified by the absence of length effects which reduces the problem from that of a three-dimensional cylinder failure to that of a two-dimensional ring failure. Soil friction angles for both wall thicknesses were backcalculated at two void ratios at the ends of the range covered in the tests. They were based

on the sample dimensions at failure which were obtained by correcting the initial sample dimensions on the basis of the measured volume changes. The rubber membranes were included in the soil dimensions in order to roughly approximate their strength. The results are presented in the following table.

TABLE K.3 MAIN STRENGTH DATA

Nominal Wall Thickness	Void Ratio 0.49		Void Ratio 0.53	
	γ	ϕ	γ	ϕ
¼"	5.5	40.9	4.58	39.4
½"	18.5	41.4	11.8	39.1

From a single exploratory test on a sample with a one inch wall thickness (void ratio = 0.50), the friction angle was backcalculated to be 40.2° (γ = 78.5). The agreement of this number with the data in the table is remarkable.

The agreement between the backcalculated friction angles for the two wall thicknesses is very good at both densities. If one tries, however, to compare these friction angles with friction angles of well-graded Ottawa Sand obtained by other methods, it is seen from Figure K.9 that the latter are lower by several degrees, the difference remaining approximately constant over the density range.

It is hard to find the right explanation for this phenomenon. Testing of samples of different lengths and different wall thicknesses made it possible to eliminate a number of possible causes of the difference, such as the influence of end supports, systematic errors in determining the void ratio, and probably the influence of the strength of the membranes (though small effects of the membrane strength might be included in the result). The only other possibilities the writer is aware of are an increase in the angle of friction of the material in the ring compression condition,

and failure of the theoretical relationship to describe the failure condition accurately.

There is considerable evidence that the theoretical relationship is correct. It evolved by integration from that condition of plastic equilibrium all through the cylinder wall which resulted in maximum ring compressive strength, and the observed phenomena at failure indicated indeed that collapse occurred due to exhaustion of the compressive strength of the cylinder wall. The consistency of backcalculated friction angles for varying ring thicknesses is another indication that the theory is correct.

From the preceding discussion it must be concluded that the calculated friction angles are indeed the true friction angles mobilized in the soil at failure, i.e., they do not represent a strength increase due to other causes.

Despite all of the past research into the strength behavior of sands, there is still a great deal about this subject which is not known. It has been established that the greater friction angle of a dense sand (as compared to a loose sand) is quantitatively related to the work which must be done to expand the dense sand. The implication is that it is easier, when shearing a dense sand, to do the extra work necessary to expand the sand than it is to shear it at its original volume. This "dilatancy correction" actually has been studied only for triaxial and direct shear testing conditions. There are certain contradictions in the results which have been obtained, and there has been no really thorough study of the development of shear resistance in dilatant sands. It is not hard to imagine, however, that a sand has greater difficulty in expanding in a ring compression test than in a triaxial or direct shear test, and that the shear resistance is correspondingly greater. Recent tests at the University of London have resulted in larger friction angles for "plain-strain" triaxial tests than for conventional triaxial tests. Also, it has frequently been observed that the friction angles of sands, backfigured from small plate bearing tests, do not agree with the friction angles as

measured in triaxial or direct shear tests.

In summary, all the available evidence indicates that the theoretical relationship developed in Section K.2 correctly describes the observed phenomena, but the friction angle to be used is considerably higher than that obtained from triaxial tests on the same soil at the same density. All other possible explanations which might bridge the gap between theory and experimental data have been eliminated, such as effects of sample length, systematic errors in the calculated void ratio, and the effects of membrane strength.

K.6.2 Volume changes

Comparison of the plots of volume changes at failure versus void ratio for the two wall thicknesses in Figures K.5 and K.7, shows the following:

Qualitatively, the trend of observed volume changes is the same at both wall thicknesses. With decreasing void ratio, the outside volume change at failure decreases and the inside volume change at failure increases. The results agree with common sense considerations. A loose sample deforms mainly by being compressed from the outside as outside pressures are applied, without too much deformation on the inside. A dense soil cylinder, on the other hand, due to its rigidity undergoes only small outside deformations as outside pressure is applied, but has to displace into the cavity in order to dilate.

Quantitatively, the results do not seem to agree too well. According to the data, the critical void ratio (i.e., the initial void ratio, for which the sample has the same volume initially and at failure) is about 0.50 for the $\frac{1}{2}$" cylinder wall and 0.52 for the $\frac{1}{4}$" cylinder wall. Also the relative slopes of the volume change curves disagree somewhat. Differences of the strain conditions in the geometrically different samples are certainly partly responsible for the observed discrepancies, and these differences team up with the scarcity* and relative inaccuracy of

* For each volume change curve, there are only half as many data points as for the strength relationship curve.

the data to render the formulation of a reliable correlation between the volume changes of the different sized samples impossible at this time. Nevertheless, a general idea of the magnitudes of deformation required to develop the compressive strength of the soil ring can be obtained. The reduction in inside diameter, for instance, might vary between a minimum of 0.013" for $\frac{1}{4}$" wall thickness and relatively loose soil to a maximum of 0.035" for $\frac{1}{2}$" wall thickness and dense soil; the corresponding radial strains are 1.3% and 3.5% respectively. These are very large numbers, if one thinks of the soil ring as acting together with a structural lining.

K.6.3 Premature failure

In the strength plots, the common lines of premature failure and occurrence of sudden volume changes are of different character for the two wall thicknesses. For $\frac{1}{4}$" wall thickness (Figure K.4), the line is horizontal, i.e., the pressure ratio is independent of void ratio. For $\frac{1}{2}$" wall thickness (Figure K.6), the line is sloping down to the right almost as much as the main strength line. Backcalculating the soil friction angles corresponding to the observed pressure ratio, as was done for the main strength lines, one finds:

TABLE K.4 PREMATURE FAILURE DATA

Nominal Wall Thickness	Void Ratio 0.49		Void Ratio 0.53	
	γ	ϕ	γ	ϕ
$\frac{1}{4}$"	3.55	36.0°	3.55	36.4
$\frac{1}{2}$"	13.6	39.8°	8.0	36.8

The agreement of the friction angles for the loose soil is very good; for the dense soil not good. Plotted in Figure K.9, the curves are seen to be located between the curves representing the main strength lines and the triaxial test data. Beyond that, however, not much more can be said

about their general location. In the investigated cases, they seem to lie an average of about 3 degrees below the friction angles of the main strength lines, with the values varying between two and five degrees.

K.7 Conclusions

K.7.1 Strength

The strength of hollow soil cylinders, symmetrically loaded at a very slow rate, can be expressed in terms of the ratio of outside pressure to inside pressure at collapse of a cylinder with a given geometry. The ratio, using medium-dense to dense Ottawa Sand, varies between 4 and 5.5 for a wall thickness of half the inside radius, and between 12 and 20 for equal soil thickness and inside radius, depending on the exact density. No significant influence of the length of the cylinder could be observed.

A general strength relationship:

$$\gamma = \left(p_o/p_i\right)_{failure} = \left(r_o/r_i\right)^{\frac{1-\lambda}{\lambda}} \qquad \text{with } \lambda = \frac{1 - \sin\phi}{1 + \sin\phi} \qquad \text{(Eqn. K.3)}$$

was derived theoretically for compression failure of a soil ring. It explained the differences in strength with density change and change in ring geometry very well, except that the soil friction angles used for best correlation were considerably higher at a given density than the friction angles obtained by triaxial tests. The change in friction angle is most probably caused by the entirely different conditions of load application and straining in the two cases.

K.7.2 Deformations

Deformations of the cylinders, in terms of radial displacements of both the outside and the inside surfaces, were observed. These displacements indicated the possible ranges of these deformations and their dependence upon soil density and ring geometry. Most important for application to the interaction of soil-structure systems, they showed that a

significant decrease of the inside diameter (1 to 4%) is necessary to mobilize the full strength of the soil ring.

K.7.3 Time effects

Variation of the rate of load application yielded interesting results: If the cylinders were loaded at a loading rate faster than a critical rate (which was of the order of magnitude of 6 minutes for the last 50% of the load), they failed prematurely, at failure pressure ratios reduced by 20 to 40%. If the samples were loaded at an intermediate rate, they experienced a sudden deformation at a load about equal to the failure load of the fast tests, but they did not fail and went on to sustain load increases up to the limit established by slow tests. Only quite slow tests (more than 15 minutes for the last 50% of the load) showed no discontinuity in the curve of volume changes versus pressure ratio.

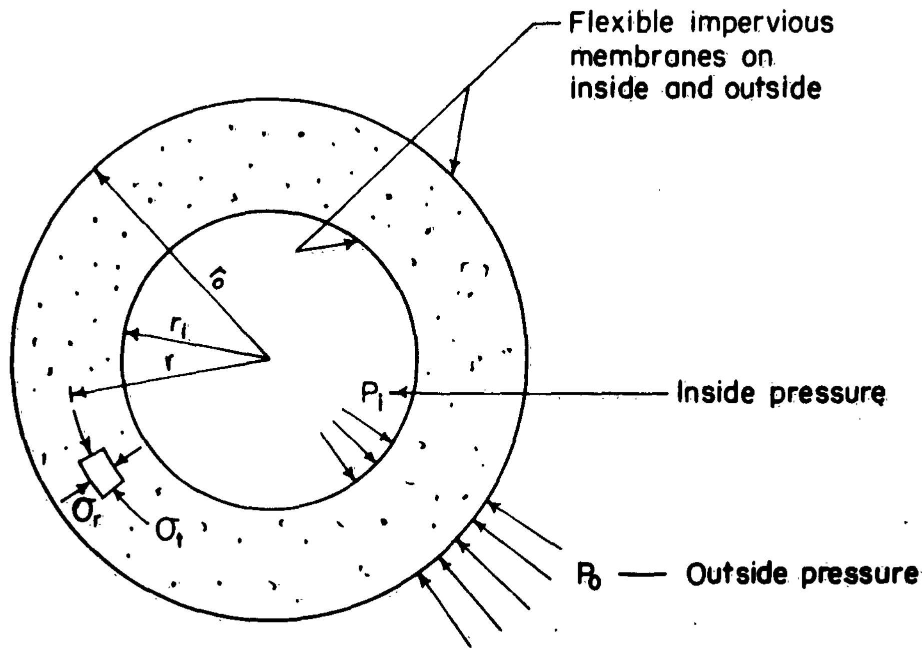


FIGURE K.1 TESTS ON HOLLOW SOIL CYLINDERS

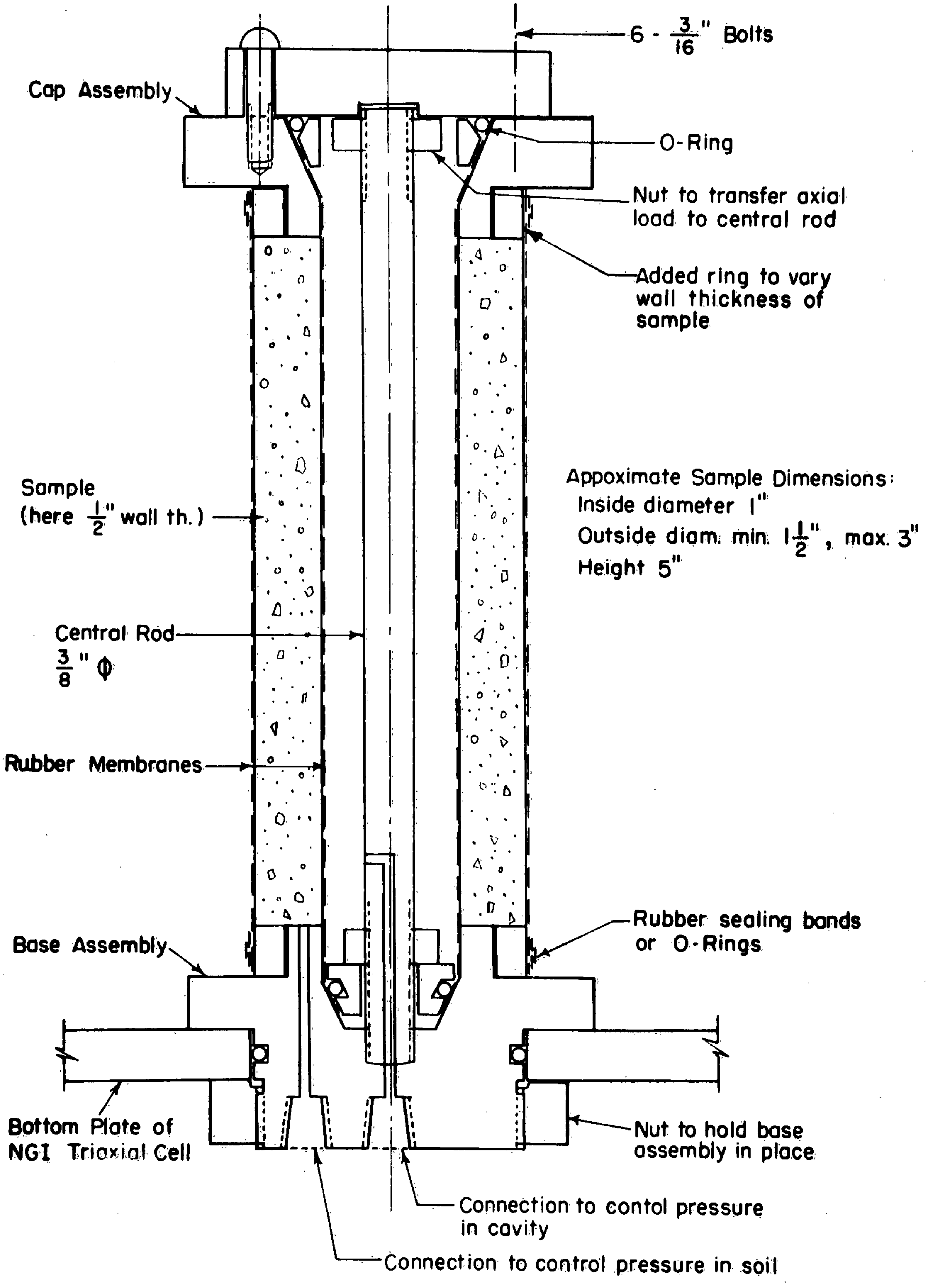
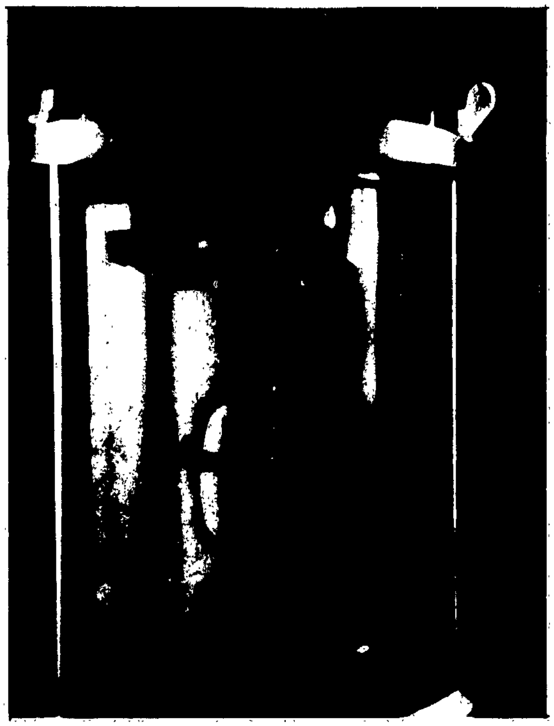


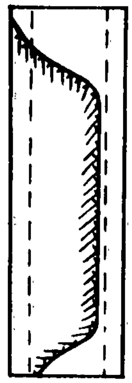
FIGURE K.2 TESTING DEVICE



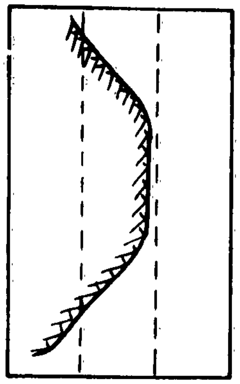
Failed "long" sample of $\frac{1}{4}$" wall thickness. Shown also is a gadget attached to the sample in an unsuccessful attempt to measure diameter changes under loading.

Failed "short" sample of $\frac{1}{2}$" wall thickness.

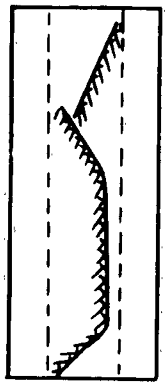
Figure K.3a FAILED SAMPLES



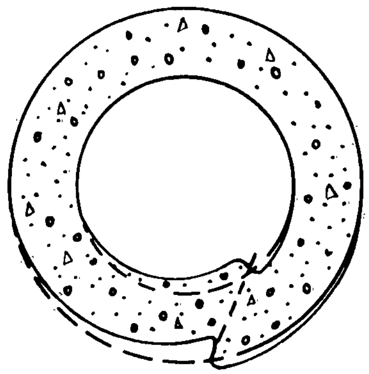
Typical failure of a $\frac{1}{4}$" - wall "long" sample



Failure of the 1"- wall sample



Typical failure of a $\frac{1}{2}$" - wall "long" sample



Cross section through typical failed sample

FIGURE K.3b SKETCHES OF FAILED SAMPLES

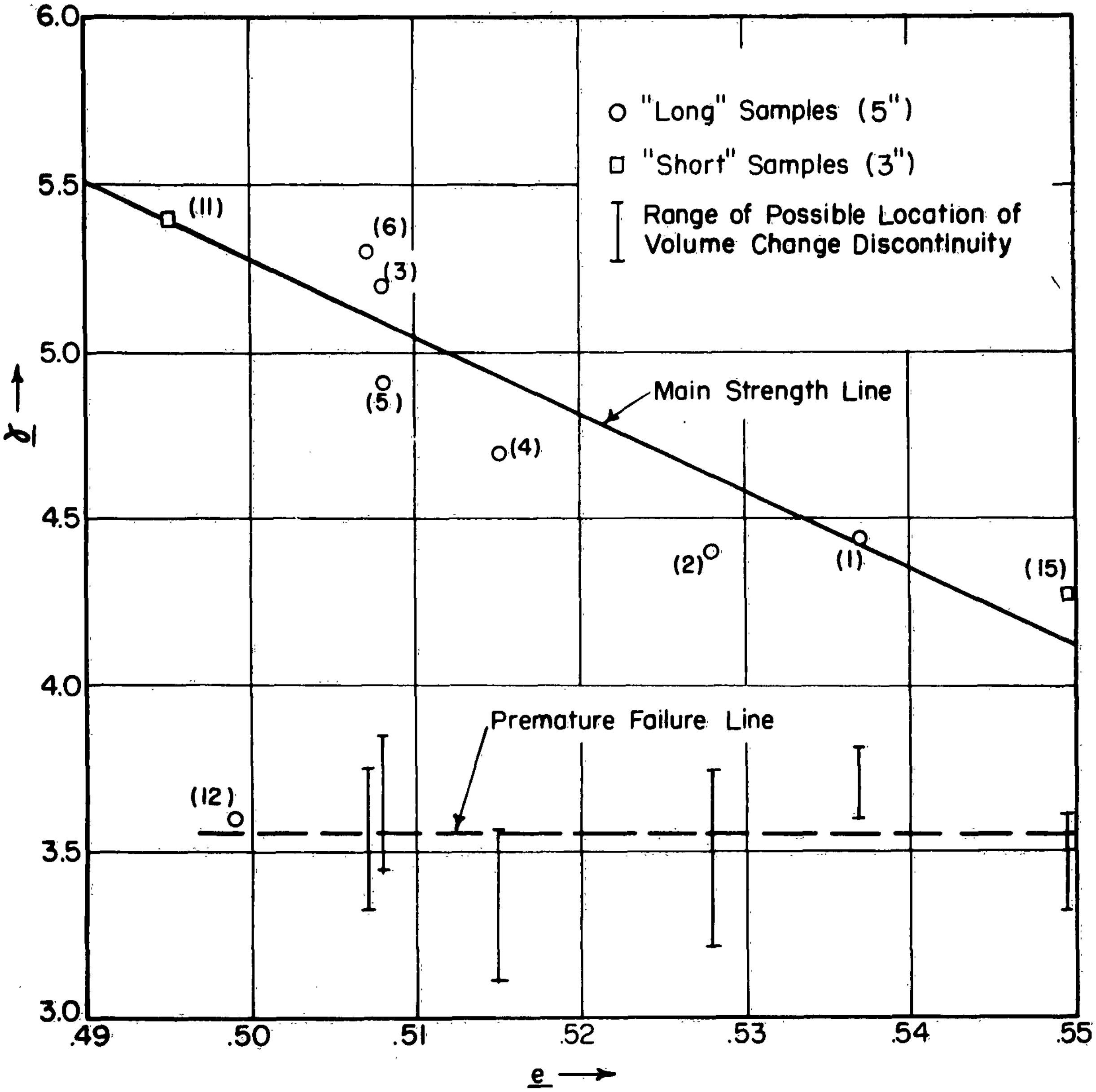


FIGURE K.4 STRENGTH DATA OF $\frac{1}{4}$" - SAMPLES

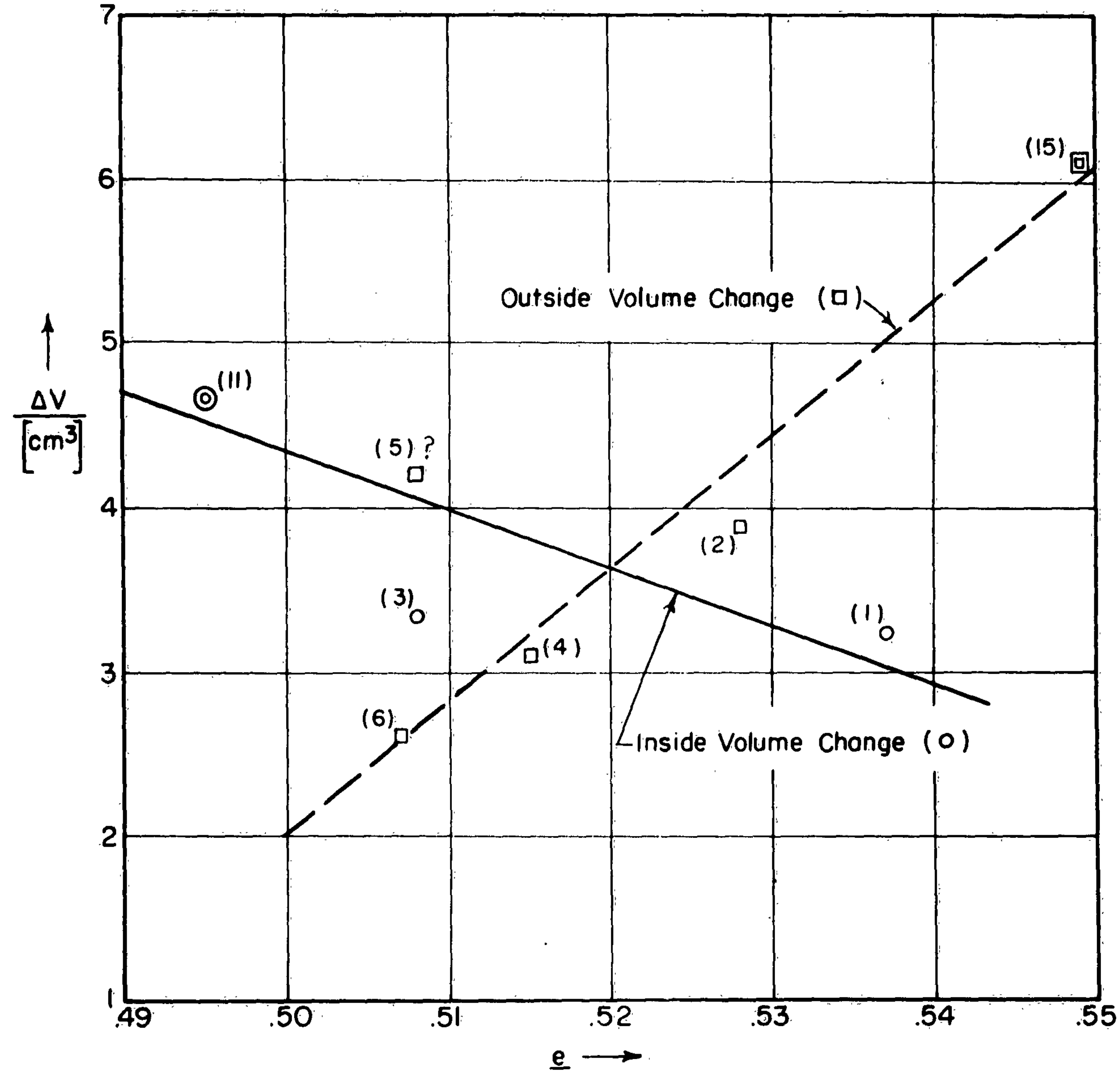


FIGURE K.5 VOLUME CHANGES AT FAILURE OF $\frac{1}{4}$" - SAMPLES NORMALIZED FOR 5" SAMPLE LENGTH

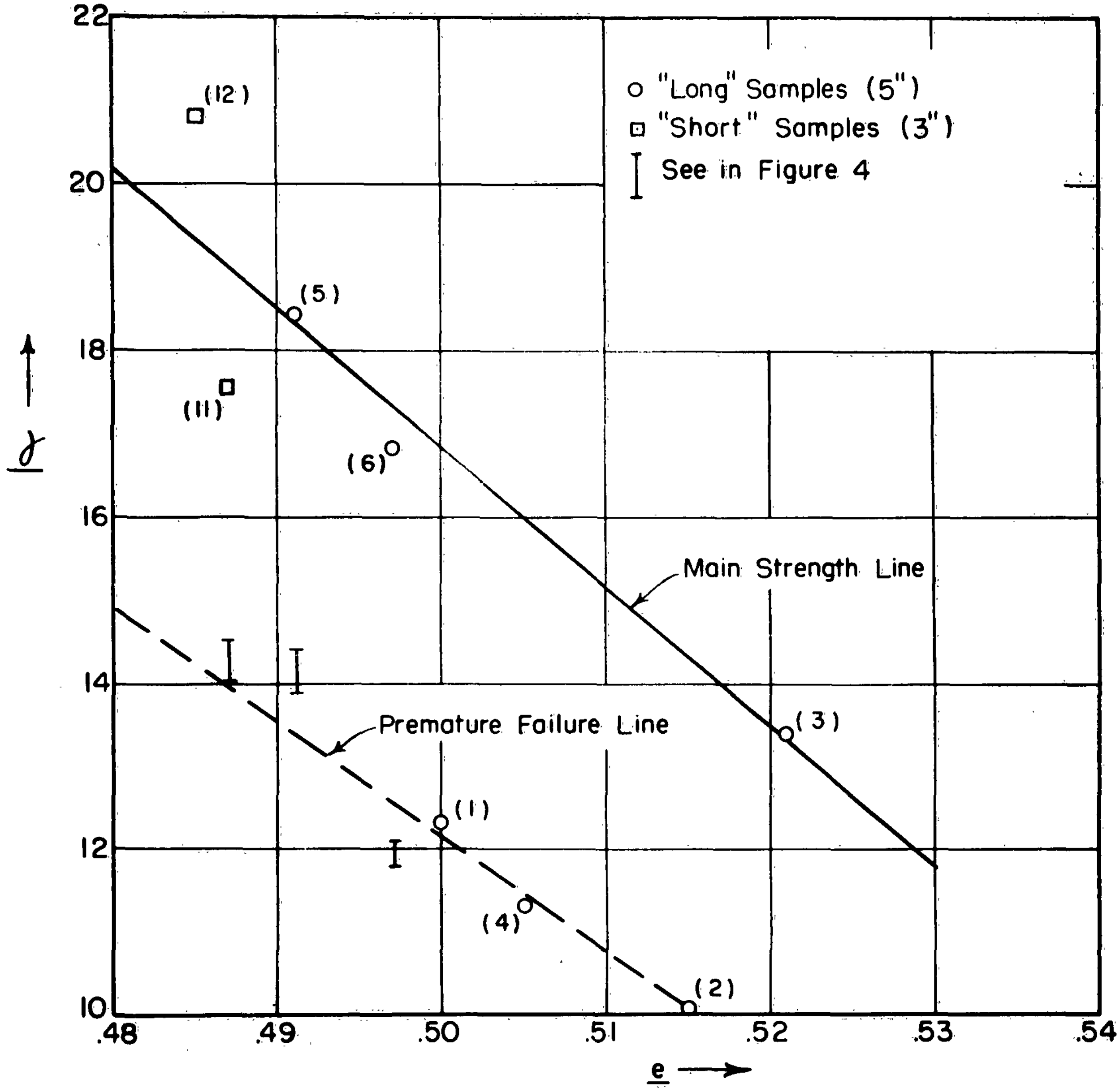


FIGURE K.6 STRENGTH DATA OF $\frac{1}{2}$" - SAMPLES

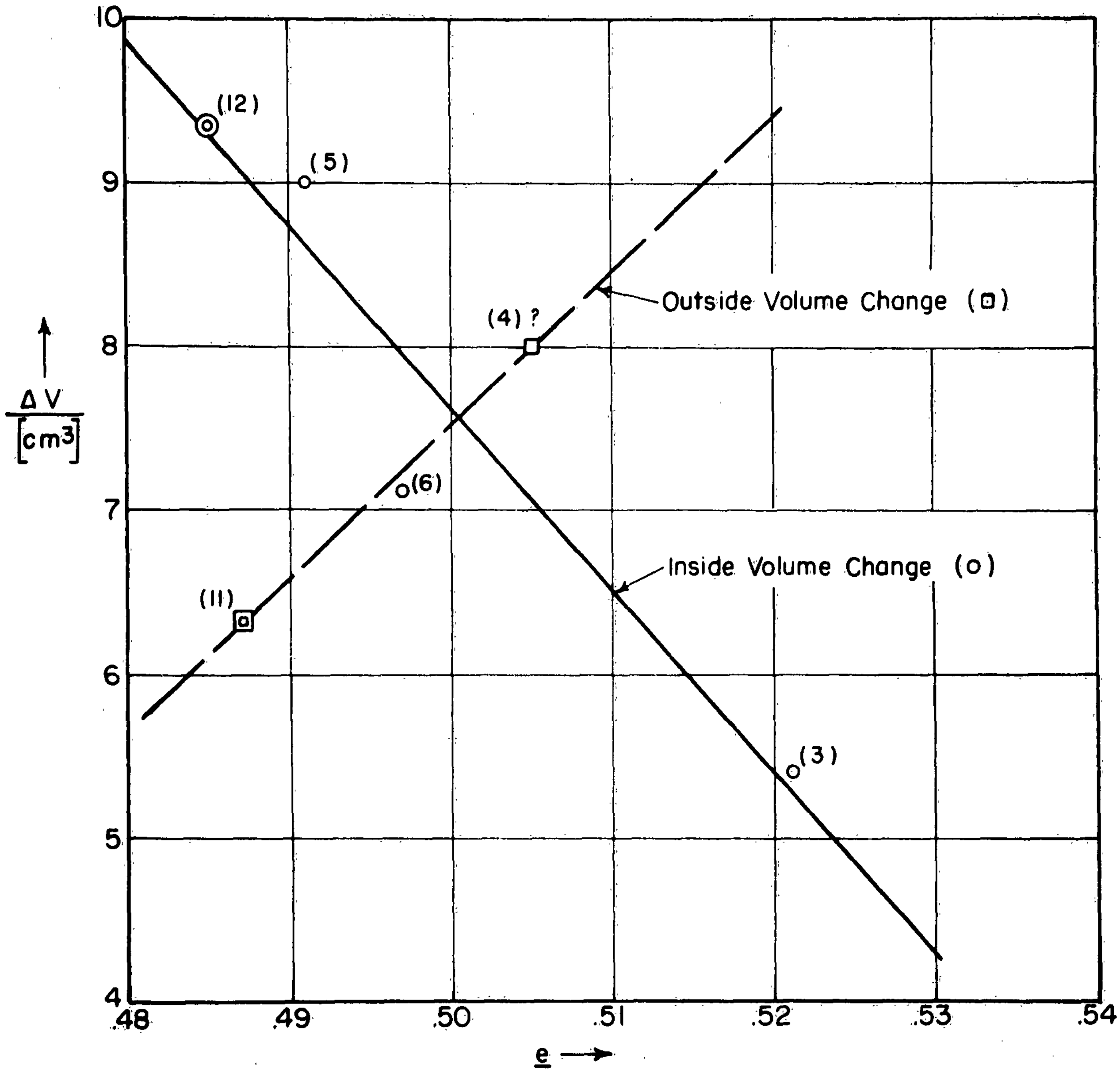


FIGURE K.7 VOLUME CHANGES AT FAILURE OF $\frac{1}{2}$" - SAMPLES NORMALIZED FOR 5" SAMPLE LENGTH

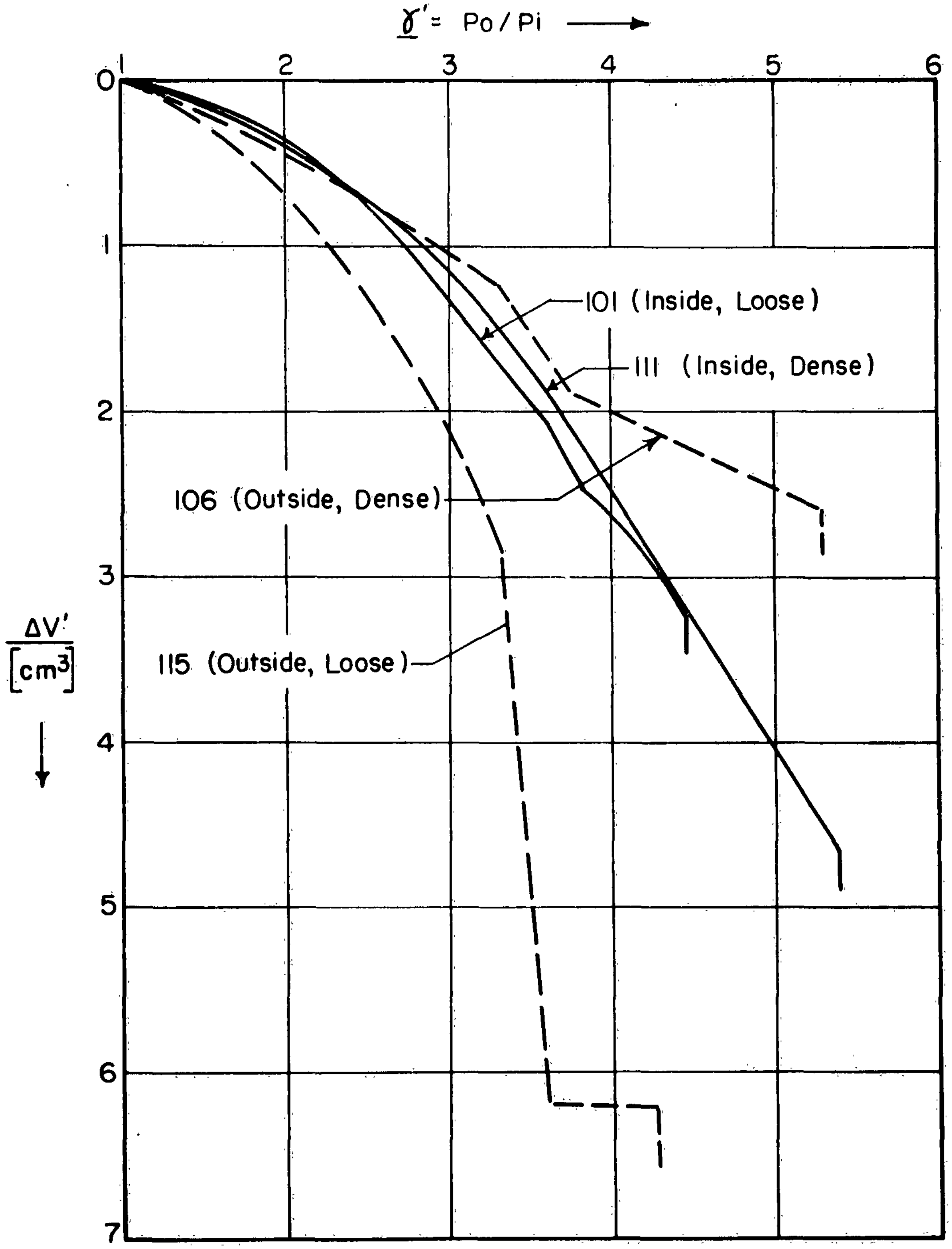


FIGURE K.8 TYPICAL VOLUME CHANGES DURING TEST DATA OF $\frac{1}{4}$" SAMPLES

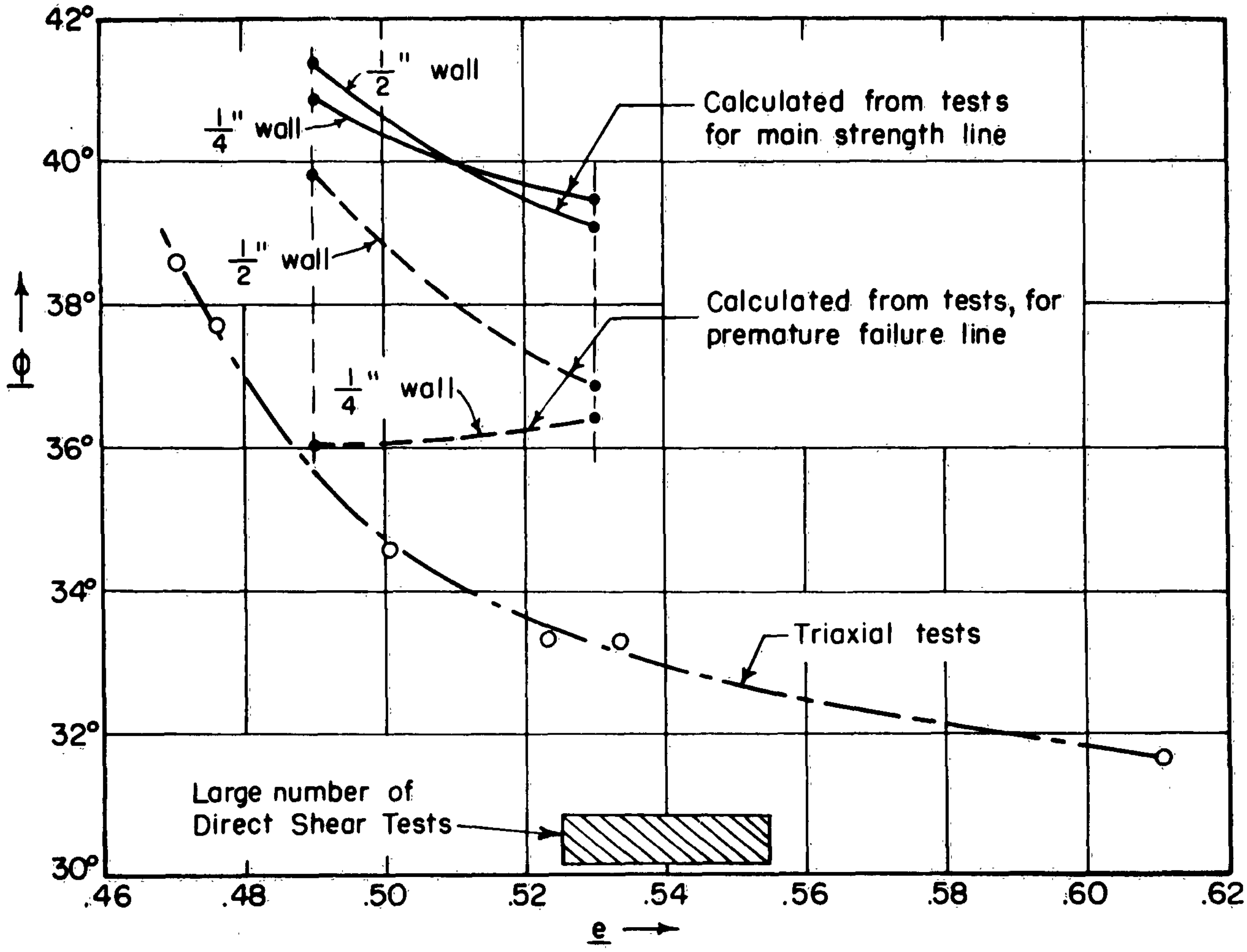


FIGURE K.9 FRICTION ANGLES

LIST OF SYMBOLS

e = void ratio

p_i = inside pressure at failure

p_o = outside pressure at failure

r = variable radius between r_i and r_o

r_i = sample inside radius

r_o = sample outside radius

ΔV = volume changes at failure

$\gamma = \frac{p_o}{p_i}$ = pressure ratio at failure

σ_1 = major principal stress

σ_3 = minor principal stress

σ_r = soil stress in radial direction

σ_t = soil stress in circumferential direction

ϕ = friction angle

λ = principal stress ratio at failure = $(\frac{\sigma_3}{\sigma_1})$

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Appendix L

TESTS UPON SOIL-SURROUNDED FLEXIBLE TUBES

Appendix L

TESTS UPON SOIL-SURROUNDED FLEXIBLE TUBES

L.1 Introduction

The tests described in this appendix resulted from a search for the simplest conceivable soil-structure interaction experiment. It will be readily apparent to the reader that tests upon circular structures embedded in a cylinder of soil subjected to uniform radial pressures are not going to answer all of the many practical problems in protective construction design practice. However, until it is possible to explain in a rational systematic way the results of such experiments, it seems clear that genuinely rational designs will not be made for more complicated situations.

L.2 Summary of previous tests at M.I.T.

L.2.1 Alumimum tube tests

The initial study was begun during the summer of 1959. Tubes employed in this first series were extruded alumimum tubes 2.0 inches in diameter and having a wall thickness of 0.035 inches. The theoretical buckling strength of an infinitely long tube of this material was computed after TIMOSHENKO (1936) to be 121 psi. For a tube of a finite length of 12 inches this strength would increase to 160 psi.

In view of these strengths a pressure vessel was fabricated to withstand a working pressure of 600 psi and a total of 5 tests were carried out on plain (i.e., no soil surrounding) tubes with SR-4 strain gauges attached. The tests were only partially successful because of failures of various parts of the apparatus as a result of the high pressures. No consistency in buckling strength could be obtained, nor did the buckling strength of the tubes appear to be a function of tube length as it should be. The strain reddings obtained could not be interpreted properly because there was no consistency in their location with respect

to the eventual buckling points. One test was attempted with 0.4 inches of sand around a tube but could not be carried to destruction because of failures in the apparatus.

The inconsistencies of these tests could be attributed to variations in the initial wall thickness of the tubes resulting from extrusion processing. The tests also demonstrated the difficulties of testing at pressures in the 100 to 200 psi region. It was recognized that tubes with high buckling strengths would require thick soil layers around them before the soil would appreciably inhibit buckling, and such combinations could be expected to reach prohibitively high pressures before failure. The obvious conclusion was that a different type tube should be obtained which would be considerably less strong and at the same time have minimum variations in initial dimensions.

During the fall of 1959 a study was undertaken to obtain a more suitable tube for testing. The original tubes (which were the thinnest walls available commercially) were turned down to 0.020 inches. At this thickness the buckling pressure of a 12 inch long tube would be 43 psi, still rather high, and the variation in thickness was excessive.

L.2.2 <u>Plastic tube tests</u>

The study was then directed toward plastic tubes of various types. Polyvinyl Chloride (PVC) and Lucite tubes were considered. Both of these types are formed by extrusion and therefore suffer the same variations in thickness as the aluminum tubes. In addition, the tubes were available only in relatively thick wall sizes and would have required considerable machining.

By far the best tubes discovered were phenolic tubes. These tubes are manufactured by a rolling process which laminates paper or fabric with a phenolic resin. The variations in wall thickness and diameter were found to be practically zero and the tubes were available in a wide range of diameters with wall thickness as low as 1/32 of an inch.

Tests employing phenolic tubes were not undertaken until the spring of 1960. Nine tests were conducted in which tubes, 1 5/8 inches in diameter and 7 1/4 inches long, were used. By themselves the tubes would buckle at approximately 14 psi. With a layer of Ottawa Sand only 0.2 inches thick surrounding them, the tubes withstood pressures as high as 300 psi without failure. In those tests where failures did occur (generally in the 200 to 300 psi region), it was suspected that rupture of the latex membrane on the outside of the sand was the cause.

No work on this subject was conducted during the summer of 1960. The continuation of soil-surrounded tube tests was resumed in the fall of 1960 and is described in the following sections.

L.3 Theory

L.3.1 Tube failure theory

Failure criterion for the tubes is buckling under lateral loading. The compressive stress in the wall is checked to ascertain whether or not it exceeds the proportional limit. For the materials and geometry used in this work, the compressive wall stress is non-critical, as it is only about 5% of the proportional limit at the maximum.

Buckling pressure for a ring under compression (and hence for an elemental section of an infinitely long tube) is given by the following formula (TIMOSHENKO, 1936):

$$q_{cr} = \frac{E\,h^3}{12 r_o^3 (1 - \nu^2)} \qquad \text{(Eqn. L.1)}$$

where q_{cr} = buckling pressure

E = Young's Modulus

ν = Poisson's Ratio

h = ring thickness

r_o = outside radius

This same formula can be applied also in the case of a tube with some end constraint if the length/diameter ratio of the tube is so large that the stiffening effect of the ends can be neglected. The accepted limiting condition for this case is a ratio of 50 . If the ratio is less than 50, the end conditions cannot be disregarded and calculations of the intensity of lateral pressure at which buckling occurs must be based on the general equations of deformation of a cylindrical shell. The development of these equations can be found in complete form on pages 446 through 453 of reference (2). The final equation for buckling pressure is:

$$q_{cr} = \frac{E\,h}{a(n^2 - 1)\left(1 + \frac{n^2L^2}{\pi^2a^2}\right)^2} + \frac{E\,h^3}{12a^3\,(1 - \nu^2)}\left[n^2 - 1 + \frac{2n^2 - 1 - \nu}{1 + \frac{n^2L^2}{\pi^2a^2}}\right]$$

(Eqn. L.2)

where: a = outside radius
L = unsupported length
n = number of buckling modes

In the experimental work the buckling pressures were measured for various L/a values, and a value of E computed as a basis of comparison to theory. The number of buckling modes is a function of the length/diameter ratio and will always be that value which gives the smallest q_{cr}. In the reference previously mentioned, values of n have been plotted in terms of dimensionless ratios involving tube properties and geometry. Observed buckling characteristics can be compared to these theoretical values.

L.3.2 Discussion

Failures of hollow soil cylinders have been given considerable attention in recent research at M.I.T. Cylinders of well-graded Ottawa sand of various lengths and wall thicknesses have been studied. Measured values of external and internal pressures at failure have been inserted into both the soil failure theory, developed in Appendix K, and the tube failure theory, presented here , and the corresponding λ - values and friction angles computed.

The tube failure theory, in general and in particular for thicker walled samples, produces friction angles which are considerably smaller than accepted values for Ottawa sand. It can be demonstrated that, under the conditions of this theory, failures will not occur in thick-walled samples at normal friction angles, a condition which simply is not correct.

Friction angles computed by the soil failure theory have been consistently too high, but have always been consistent for various wall thicknesses. At first the high values were attributed to the effect of end restraint. However, recent experiments to study end effects have given strong evidence that there is no measurable change in soil cylinder strength as a result of end restraint. The weight of experimental evidence now indicates that the soil failure theory is correct and the high friction angles are a result of the unusual strain conditions of the soil ring.

L.4 Procedure

L.4.1 Selection of a tube for testing

In the fall of 1960 the same problem which had been encountered a year before was again preventing continuation of tube tests. Although a tube material only 1/10 as strong as the original aluminum tubes was being used, testing pressures were again causing failures in various components of the experimental apparatus.

At that time two alternatives were available. The first was to design and fabricate new apparatus capable of handling much higher pressures (up to 1500 psi). Although it would have been possible to overcome most of the difficulties inherent in the original equipment, it was felt that no solution could be found to improve the latex membranes which had proven unreliable at pressures greater than 200 psi. This fact, combined with the danger of the high pressures and the difficulties of testing "blind" in a strong pressure vessel, made this first alternative undesirable.

The second choice was to obtain weaker tubes, either by using larger tubes, or by employing a weaker tube material. Larger tubes posed problems of locating or making sufficiently large membranes, and their testing would be time consuming to carry out because of the large amount of soil which would have to be carefully placed around them. In view of the fact that a large number of tests were planned, this was an important consideration.

A new search for a weaker tube material eliminated commercial products as a source of supply. Although several types of tubes (e.g., cardboard) were located, none of them were well suited to the test requirements.

It was found that satisfactory tubes could be formed by rolling a double layer of heavy aluminum foil on a glass tube. The layers of foil were bounded together by a thin coat of shellac, and crude preliminary tests indicated that the tubes would buckle under a lateral load of something less than 1 psi. The tubes offered certain additional advantages in that they could be produced in almost any size, and their ends could be sealed with O-rings, thus eliminating the need for an inside membrane and thereby simplifying the apparatus considerably.

A series of tests which had a twofold purpose was undertaken on the aluminum foil tubes. The first was to check the reproducibility of buckling pressures. The second purpose was to study the effect of the length/diameter ratio on the buckling strength of the tubes. A total of 12 tests were run on 4 groups of 3 tubes each of lengths 15.0, 11.5, 7.5, and 5.0 inches. The tube diameter in all tests was 1.616 inches. The results of these tests are summarized in Table L.1.

Young's Modulus has been computed from the known buckling pressure and number of buckling modes for each test. The results show only fair consistency in the buckling pressures and rather poor agreement in the computed moduli. Despite these facts the decision to go ahead with foil

TABLE L.1
RESULTS OF ALUMINUM FOIL TUBE STUDIES

Test No.	Length (inches)	Failure Pressure (psi)	Corrected Pressure* (psi)	Modes	Computed Modulus (psi)	Remarks
1	15.0	0.22	0.13	3	4.7×10^6	Tube Leak Press. High
2	15.0	0.20	0.13	3	4.2×10^6	
3	15.0	0.19	0.13	3	4.0×10^6	
4	11.5	0.19	0.17	3	3.5×10^6	Seam Split Press. Low
5	11.5	0.23	0.17	3	4.2×10^6	
6	11.5	0.25	0.17	3	4.6×10^6	
7	5.0	0.39	0.40	4	2.7×10^6	
8	5.0	0.46	0.45	5	3.1×10^6	
9	5.0	0.40	0.40	4	3.1×10^6	
10	7.5	0.27	0.29	4	2.9×10^6	
11	7.5	0.25	0.29	4	3.1×10^6	
12	7.5	0.29	0.29	4	3.1×10^6	

* See Section L.4.1

tubes was made. It was later discovered that an instrumentation error had, in all likelihood, contributed to most of the error.

The interior of the tubes had been vented to the atmosphere to prevent a build-up of pressure due to tube deformation. This vent had been led into a test-tube of water to provide a check for leaks. The column of water acting on the tube was short, but could have contributed a pressure of about 0.08 psi to the tubes' interiors. This would easily account for all the variation in the tube buckling strength and computed moduli, mainly because the error was not a constant for each test.

Control tests on the tubes used in soil-surrounded tests have given excellent consistency after the elimination of this source of error, and the modulus from these tests has been used to compute what the pressures should have been in the original tests. The magnitude of the error is within the limits of the estimated experimental error for all cases.

A complete description of the method of manufacturing tubes is included in the next section.

L.4.2 Preparation of Aluminum Foil Tubes

Tubes are made of aluminum foil manufactured by the Reynolds Metals Company and marketed under the name of "Heavy-Duty Reynolds Wrap". A roll of foil was selected from stock which appeared to be especially free from wrinkles, dents, and other imperfections often observed in rolls of foil. The foil is 0.0015 inches thick and can be obtained in rolls 18 inches wide and 50 feet long.

A sharp knife and stiff straight edge give the best results in trimming the foil to size. A clean glass desk top is a good working surface for cutting and rolling. Cleanliness is important because any grit on the working surface dimples or even punctures the foil.

The foil is rolled to a double thickness on a lucite tube with an overlap of $\frac{1}{2}$ inch on the seam. Rolling is best accomplished in the same direction as the foil was originally rolled in its box. The initial roll is made with no shellac to check dimensions and evenness. Upon unrolling a fine line is discernible where the tube begins to roll over itself on the mandrel. Shellac is painted upon this line with a wide, soft brush. A thin line of shellac is painted on the inside edge of the foil where it will touch the mandrel to aid in holding the edge down as the roll is started.

Once the tube is rolled with shellac, it must be quickly removed from the mandrel to prevent its becoming stuck. The rolling must be done smoothly and evenly to assure a firm fit on the mandrel and a wrinkle-free tube. Upon removal the tubes are stored in an upright position and are given a minimum of handling.

The mandrel is a lucite tube with nominal diameter of 1 5/8 inches. It was selected from stock to be particularly uniform in diameter at all sections. It is easily cleaned with alcohol after each rolling process. A line scribed longitudinally along the tube aids in starting the foil evenly during rolling.

The tubes prepared for testing are 11 inches long, 1.630 inches in diameter, and have a wall thickness of 0.0030 inches (the shellac contributes no measurable amount to the wall thickness).

L.4.3 Apparatus

The special apparatus prepared for the soil-surrounded tube tests which is shown in Figures L.2 and L.4 consists of relatively simple parts fabricated from lucite and aluminum. The base is a lucite piece 2 inches in diameter with a raised portion the diameter of the tubes to be tested. An aluminum rod, 11.5 inches long, which serves to carry all axial stress, is screwed firmly into the center of the base. A small hole in the top of this rod serves to vent the interior of the tube through

the top cap. A nut soldered at the top of the rod insures that the top cap is in the same place during each test so that the unsupported length of the tube is a constant for all tests.

Because the tube must be supported while the soil is being placed, the top cap consists of two pieces to allow sealing after the support is withdrawn. The first piece screws down inside the tube so that the interior is sealed. The second piece seals the soil column and contains fittings for the tube interior and soil pressure vents. An O-ring prevents leakage between the soil system and the tube interior.

Lucite tubes of appropriate sizes serve as molds while the soil is being placed. These molds are cut in half longitudinally so that they can be removed during the test. The tube is supported by a lucite rod, the same one upon which it was originally formed. The end pieces can be adapted for use with any soil thickness by the addition of lucite rings which serve to increase their diameter to the proper size. The major advantage of this apparatus over that used in earlier soil-tube tests is that the pressure in the soil and the interior of the tube are isolated from each other and can be controlled independently.

The pressure cell in which tests were run is an old MIT triaxial cell, 15 inches high, modified to accommodate soil pressure and tube interior vents through its top cover. Pressure is supplied by a nitrogen tank equipped with a 0-60 psi regulator valve. Pressures up to 1.5 psi are measured on a water manometer and read to the nearest 0.01 psi. Higher pressures are measured with a mercury manometer and read to the nearest 0.05 psi. A schematic diagram of the set-up is shown in Figure L.3.

L.4.4 Preparation of tests

The soil-surrounded tubes were prepared in the following manner: A light coating of stopcock grease was placed on the bottom end piece and a tube with mandrel inside for support was slipped onto the

piece. An O-ring was placed around the outside of the tube to seal it against the end piece. A small amount of plasticine, a putty-like material, was placed around the base of the tube to insure complete sealing of the tube interior.

Next, the outside membrane was slipped onto the bottom piece and sealed with two O-rings and plasticine. The split mold was then placed around the outside of the membrane and clamped securely to the base. It was found that a light coating of talcum powder would prevent the mold from sticking to the membrane, greatly facilitating its removal later in the test. The free end of the membrane was pulled to the top of the outside mold and folded over the end to expose the interior for filling with sand.

At this point the mandrel was withdrawn and the two end pieces were screwed into place to align the tube and the exterior mold concentrically. Once aligned, the end pieces were removed, the mandrel was replaced, and the filling process was begun.

Ottawa Standard sand, used as the soil material in all tests, was poured into the membrane through a small glass funnel. When a depth of approximately one inch was reached, filling was stopped and the sand was tamped into place. The tamper was a long stiff wire with a small section of phenolic tube fastened to the base. This device proved to be very useful in pushing the sand firmly against the mold wherever the membrane tended to wrinkle. The fill-tamp process was repeated until the sand was within $\frac{1}{2}$ inch of the top of the tube. The weight of sand used was carefully measured so that the void ratio of the soil surrounding could be computed.

It was found necessary to check the concentric alignment after each layer of sand had been placed until the mold was about half full. The mandrel was withdrawn and the top pieces were used to make the check. Any adjustments had to be made with the mandrel back in place to prevent

wrinkling of the tube. Failure to obtain good alignment resulted not only in an uneven thickness of sand around the tube, but would result in the tube being wrinkled when the top pieces were put into place at the end of the filling process.

With the sand near the top of the tube the mandrel was withdrawn and the first end piece was screwed down inside the tube. This piece was sealed with an O-ring and plasticine. The sand level was then brought up flush with the top of the end piece and the second end piece was screwed firmly into place. The membrane was unfolded from the top of the mold and secured around the second end piece with O-rings and plasticine. This completed the preparation of the soil-structure system for testing.

L.4.5 Testing procedure

Four different types of tests were run and each type is described separately in the following paragraphs.

Control tests

Tests 1 through 7 were on tubes with no soil surrounding to determine their unrestrained buckling strength. The special apparatus previously described was employed to support the tube. The tube was placed in the pressure chamber, its interior vented to the outside of the cell. Pressure was then increased gradually until a buckling failure occurred. The time to failure was in the order of magnitude of a half to one hour in all tests. Four of the control tests were on tubes ranging in age from several days to two months. Three tests were performed on freshly made tubes to check for possible effects due to the drying of the shellac used to glue the tubes.

Restrained Buckling Tests

Tests 8 through 13 were prepared as described in the previous section with the exception that in some cases no membrane was used.

The samples were placed in the pressure cell with the exterior mold left in place. The interior of the tube was vented to the outside, and the pressure in the sand was open to the chamber pressure. Thus the pressure acted directly on the tube with the sand and mold serving only to restrain the buckling of the tube. Chamber pressure was increased gradually until failure occurred.

Soil-Tube Tests

Tests 14 through 26 were prepared as described in the previous section. The sand was initially placed under a vacuum of 0.5 to 2.0 psi (depending on the soil thickness), which permitted the exterior mold to be removed. The system was then placed in the pressure cell, the tube interior vented to the outside, and the cell pressure increased with the vacuum being correspondingly decreased. Once the vacuum was reduced to zero, the sand was also vented to the outside, and the cell pressure was gradually increased until failure of the soil-tube system occurred. Tests were run on samples with nominal soil thicknesses of 3/16, 3/8, and 7/16 inches.

Modified Soil-Tube Tests

Tests 27 through 29 were started exactly as were the regular soil-tube tests. The cell pressure, however, instead of being increased to failure was raised to a value less than failure pressure. The pressure in the cell was then maintained at this value, and the pressure in the sand was increased from zero until failure of the system occurred. These tests were all performed on samples with a soil thickness of 7/16 inches.

L.4.6 Additional Remarks

The soil used in all tests was Ottawa Standard Sand. Tubes were of aluminum foil as described in Section L.4.2. The unsupported length of the tubes in each test was 10.00 inches. Radial loads only were carried by the tubes and surrounding soil. All axial loads were carried by the $\frac{1}{2}$ inch aluminum rod through the center of the sample.

L.5 Results

L.5.1 Tabulation of results

Results of all tests are tabulated in Tables L.2, L.3 and L.4.

L.5.2 Test observations

Although all experimental data are included in the tables, it is felt that certain observations made at the time of testing should be included. The following descriptions and photographs are intended to provide a background to the general characteristics of the tests.

L.6 Discussion of results

L.6.1 Control tests

Excellent consistency of buckling strength of tubes was obtained in these tests. The elastic modulus computed from the buckling theory is 3.1×10^6 psi. The value generally accepted for aluminum is in the region of 10×10^6 psi. The discrepancy, however, is not serious in view of the unusual type of aluminum used, and the unusual manner in which it was loaded. As a comparison, strains measured in early tests on heavy aluminum tubes give values of approximately 4×10^6 psi for the elastic modulus.

Examination of some old tubes has indicated that the shellac used to glue the two layers together does dry out. If tubes are permitted to sit for a month or more before use there may be some changes in their properties. No differences were observed between freshly made tubes and tubes up to four weeks old. Therefore, care was taken to use only recently made tubes in all soil-tube tests.

The four buckling modes observed in these tests are in agreement with the number predicted by the buckling theory.

TABLE L.2
EXPERIMENTAL RESULTS

Test No.	Type	Soil Thickness (inches)	Outside Radius (inches)	Initial Void Ratio	Failure Pressure (psi)	Remarks
1		-	0.815	-	0.28	
2		-	0.815	-	0.26	
3		-	0.815	-	0.26	
4	I	-	0.815	-	0.26	
5		-	0.815	-	0.26	Fresh Tube
6		-	0.815	-	0.26	Fresh Tube
7		-	0.815	-	0.26	Fresh Tube
8		0.185	1.000	0.57	0.60	
9		0.185	1.000	0.54	0.80	
10	II	0.185	1.000	Dense	0.69	
11		0.185	1.000	Loose	0.88	
12		0.185	1.000	Loose	1.03	
13		0.380	1.000	Loose	0.72	

TABLE L.3
EXPERIMENTAL RESULTS

Test No.	Type	Soil Thickness (inches)	Outside Radius (inches)	Initial Void Ratio	Failure Pressure (psi)	Remarks
14		0.185	1.000	0.50	0.95	
15		0.185	1.000	0.54	1.21	
16		0.185	1.000	0.54	1.27	
17		0.380	1.195	0.56	5.35	
18		0.380	1.195	0.60	-	Test Ruined
19		0.380	1.195	0.60	4.0	Leak
20	III	0.380	1.195	0.61	6.9	
21		0.380	1.195	0.61	6.8	
22		0.430	1.245	0.54	2.8	Tube Damaged
23		0.430	1.245	0.57	6.9	Leak
24		0.430	1.245	0.57	9.0	
25		0.430	1.245	0.57	5.2	
26		0.430	1.245	0.57	9.4	

TABLE L.4

EXPERIMENTAL RESULTS

Modified Soil - Tube Tests

Test No.	Soil Thickness (ins.)	Initial Void Ratio	External Pressure	Pore Pressure in Sand
27	0.43	0.57	5.05	1.59
28	0.43	0.57	7.00	0.90
29	0.43	0.57	4.00	1.85

L.6.2 Restrained buckling tests

These tests were undertaken to study the effect of the sand around the tube on its buckling strength. The results are not particularly meaningful for several reasons. First, the restraint was far from perfect because the sand had little or no shear strength. Buckling was obviously inhibited, but it would be impossible to determine how effective the sand really was. A second consideration is the obvious dependence of strength on the amount of compaction given the sand. The more compaction given the sand, the more rigid it would have been, which should have given higher instead of lower buckling strengths. The implication is that the sand itself was exerting forces on the tubes which were contributing to the failures. Once again, it would be impossible to determine the exact contribution of these forces.

These considerations explain the wide variations of buckling strengths observed in these tests. Although the results cannot be properly analyzed, they clearly demonstrate the effect of a surrounding soil medium on the buckling properties of the tubes. Later tests have indicated that these tests were realizing less than half the potential of the sand to restrain the buckling.

L.6.3 Soil-tube and modified soil-tube tests

Considerable problems were experienced in reproducing data in these tests. In several instances there are possible reasons why low strength values were obtained; these are mentioned in the remarks section of Table L.3. In many cases no immediate explanation is available for the low strength values observed. Whatever the problem may be, however, it is felt that the highest values observed are the measure of the true strength potential of each soil-tube system. For each thickness the strength value used was obtained in at least two tests, and it is felt that there is ample justification for discounting all low values in an initial analysis.

The results of the modified soil-tube tests are basic to the

analysis of the work done. These results have been plotted in Figure L.1. The abscissa is the net pressure acting on the whole system (i.e., external pressure minus pressure in sand) and the ordinate is the pressure in the sand which acts directly on the tube. Included on the plot is a point from test number 26 which corresponds to having zero pressure in the sand.

As can be seen, all the points lie on a straight line which, when projected, passes through the value of 2.5 psi on the ordinate. This point represents a failure condition of the tube in which all the pressure acting on the tube is pore pressure in the sand. This point cannot be reached experimentally because there must be an effective stress in the sand for it to stand up. The point may be thought of as the failure point of a tube, with buckling restrained, subjected to only a lateral air pressure. Hence the value of 2.5 psi can be considered the maximum restrained buckling strength of the tubes.

L.6.4 Analysis of results

As a starting point for the analysis, it will be assumed that the failure of the soil-tube systems can be analyzed on the basis of the soil ring failure theory by inserting values of the tube strength for the internal pressure acting on the soil ring. If such an analysis produces reasonable answers, the implication will be that the presence of the tubes does not modify the failure conditions of the soil ring.

The soil failure theory, as discussed in Appendix K has been chosen as the best of the two theories presented. It defines the failure condition of a soil ring as:

$$\gamma = \left(\frac{r_o}{r_i}\right)^{\frac{1-\lambda}{\lambda}} \qquad \text{(Eqn. L.3)}$$

where: $\gamma = \dfrac{p_o}{p_i}$ and $\lambda = \dfrac{1 - \sin\phi}{1 + \sin\phi}$

Consider the results of test number 26 in Table L.3. Outside pressure is 9.4 psi, outside radius is 1.245 inches, and inside radius is 0.815 inches. Assuming the internal pressure is 2.5 psi, γ is 3.76. Inserting these values into the formula:

$$3.76 = \left(\frac{1.245}{0.815}\right)^{\frac{1-\lambda}{\lambda}} \qquad \text{(Eqn. L.4)}$$

and solving for λ :

$$\lambda = \frac{\log\left(\frac{1.245}{0.815}\right)}{\log 3.76 + \log\left(\frac{1.245}{0.815}\right)}$$

$$\lambda = 0.232 \text{ and } \phi = \sin^{-1} \frac{1-\lambda}{1+\lambda} = 38.5^{\circ}$$

A friction angle of 38.5° is high for Ottawa Sand, but as was stated in section 3.2, values computed by the theory have been consistently high, so certainly this cannot be considered unreasonable.

Continuing the analysis to the 3/8 inch wall tests, it would seem entirely reasonable to expect the maximum tube strength to be the same for these tests. Now that a friction angle has been established, it is possible to compute what the internal pressure on the soil ring must have been.

The friction angle must be adjusted to account for the difference in void ratio of the two tests. This is accomplished by referring to Figure 14.10 on page 349 of reference (1) which plots friction angle versus void ratio of Ottawa Sand. For a friction angle of 38.5° at an e of 0.57, the corresponding friction angle for an e of 0.61 is 35.5°.

With this value of ϕ it is now possible to compute a value of p_i as follows:

$$\lambda = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.266 \qquad \text{(Eqn L.5)}$$

$$\gamma = \left(\frac{1.195}{0.815}\right)^{\frac{1-.266}{.266}}$$

$$\gamma = \frac{\log\left(\frac{1.195}{0.815}\right)}{.266} = \log\left(\frac{1.195}{0.815}\right)$$

$$\text{or } \gamma = 2.86 = \frac{p_o}{p_i}$$

$$\text{but } p_o = 6.9 \text{ psi, so } p_i = \frac{6.9}{2.86} = 2.41 \text{ psi}$$

This result is completely in line with the expectations.

A similar analysis can be done for the 3/16 inch wall samples. In this case it is difficult to anticipate the internal pressure. It certainly will not be 2.5 psi because the external pressure is only 1.27 psi (test number 16). Carrying the analysis out, the friction angle is adjusted to 40°, and the internal pressure turns out to be 0.61 psi which can be favorably compared to values obtained in the restrained buckling tests.

Strong support of this method of analysis is obtained by examination of the modified soil-tube tests. The theory can be used as in the preceding analyses to compute the pressure which the soil must have been exerting on the tube. This value added to the pore pressure in the soil should be equal to the maximum restrained buckling strength of the tubes.

The friction angle and ratio of radii are the same for each of these tests as in test number 26. Therefore, γ will be the same or 3.76. The external pressure in these cases is the net pressure acting on the soil, and the internal pressure is computed by dividing the net pressure by γ. The internal pressure is the pressure which the tube must have exerted on the soil and conversely the soil on the tube. This value is added to the pore pressure in the sand at failure to find total pressure acting on the tube at failure. The results are summarized in the following table.

TABLE L.5 ANALYSIS OF RESULTS OF MODIFIED SOIL-TUBE TESTS

Test Number	Net Pressure	Internal Pressure	Pore Pressure	Tube Pressure
27	3.46	0.92	1.59	2.51
28	6.10	1.66	0.90	2.56
29	2.15	0.57	1.85	2.42

The values of the pressure acting on the tube check the assumed value of 2.5 psi almost perfectly.

All of these analyses indicate quite strongly that the failure theory for a soil ring may be superimposed in the soil-tube system by substituting a tube strength for the interior pressure acting on the soil ring.

L.6.5 Discussion of tube strength

The foregoing discussion indicates that almost a ten-fold increase in tube strength was obtained by surrounding it with a relatively thin layer of soil. This increase was expected, but only qualitatively; there was no way of anticipating the increase quantitatively. Now that the increase is known quantitatively, an important consideration is an explanation of the observed value.

It is clearly understood why the strength increases -- the surrounding soil simply prevents buckling of the tube in its usual manner. The most logical way to analyze this phenomenon quantitatively is to assume that the surrounding soil forces the tubes into higher modes of buckling. That is to say that a tube will fail at a pressure which is capable of forcing it to buckle in some higher mode which, owing to its smaller deformations, the soil is unable to restrain. Thus a tube which normally buckles with four modes may be forced by the soil restraint into eight modes with a corresponding increase in strength. All eight modes would not be likely

to appear because as soon as one mode began to form, the whole soil-tube system would collapse.

With this possibility in mind, buckling strengths of the experimental tubes for higher modes of buckling have been computed. These values were obtained by inserting various values of n into the buckling formula presented in Section 3.1 and solving for q_{cr}. The results are as follows:

TABLE L.6 BUCKLING STRENGTHS OF BARE TUBES

Number of Modes	Buckling Strength (psi)	Number of Modes	Buckling Strength (psi)
4	0.26	9	1.39
5	0.42	10	1.71
6	0.61	11	2.08
7	0.83	12	2.48
8	1.09	13	2.91

As can be seen, the buckling strength with 12 modes is very close to the value assumed for the restrained buckling strength of the tubes. This, by itself, could be a coincidence, but other data indicates that it is not. For example, in Section 6.4 the pressure acting on the tube at failure in test number 16 was computed to be 0.61 psi, corresponding exactly to 6 modes of buckling.

Even better indications may be found by examining those tests which for no apparent reason failed to attain strengths as high as similar tests. A reasonable explanation would be that they failed to reach as high a buckling mode as the stronger tests. If this is true, the pressures acting on the tubes at failure should correspond to a buckling strength of some lower mode. Calculations have been made for these tests and the results are tabulated in the following table.

TABLE L.7 ANALYSIS OF EXCEPTIONAL RESULTS OF SOIL-TUBE TESTS

Test Number	Failure Pressure	γ	Pressure on Tube	Corresponding Modes	
14	0.95	2.21	0.43	5	0.42
17	5.35	3.67	1.46	9	1.39
19	4.0	2.76	1.43	9	1.39
23	6.9	3.76	1.83	10	1.71
25	5.2	3.76	1.38	9	1.39

All but one of the tests are in good agreement with buckling strengths previously computed. In addition to supporting the proposition that the tubes gain strength by buckling in higher modes, the calculations provide a logical explanation of why low strength values were observed in seemingly normal tests; the tubes in these tests simply buckled with fewer modes.

There appears to be no way to predict the number of modes the tubes may be forced to assume before buckling. The number apparently is not a function of the effective stress in the sand, because in the modified soil-tube tests the effective stress varied from 2.15 to 6.1 psi and yet the tubes all buckled at the same pressure. In tests in which all measurable quantities appear to have been the same, the tubes buckle sometimes with 9 modes and sometimes with 12. It is not difficult to imagine that imperfections created during the preparation of the soil-tube tests could result in the variation observed. It is felt, however, that more experimental work would be required to isolate and analyze those factors which determine the failure configuration of the tubes.

L.6.6 Strain considerations

Throughout the foregoing discussion it has been implicitly assumed that the strain conditions in the soil ring and in the tubes were compatible. However, rough volume change measurements made during the

failure of hollow soil cylinders indicate that this is not true. In fact, it has been calculated that the strain in the soil ring must be some 1000 times as great as the strain in a tube with 2.5 psi pressure acting on it. Even allowing for the fact that the measurements of volume change were very rough, these values represent a large discrepancy in the superposition of ring failure theory into the soil-tube system.

In attempting to explain the discrepancy, there are only two basic conditions possible. The first of these is that the shape of the tube was being modified to conform to the strains in the soil while the tube somehow managed to retain its ability to support a load. This possibility is unlikely for several reasons. It is hard to imagine the thin tubes undergoing any drastic strains and still being able to support any sort of load. In addition, the consistency of the tube strengths compared to theoretical buckling strengths must be considered. If the shape of the tube were modified so much by the strain occurring in the soil, certainly there would not be any comparison of tube strength and theoretical buckling strength.

It might be possible to develop a local buckle in the tube to accommodate the large strains with the soil pressure on the tube being carried by an arch of soil across the crease. If this were true, however, in the modified soil-tube tests the pore pressure introduced into the sand would act directly on the crease and would result in immediate failure of the tube.

A final factor is the amount of air observed escaping through the tube interior vent. No quantitative measurements have been made, but it can be stated that the amount observed escaping during soil-surrounded tests was the same order of magnitude as the amount observed during tests on tubes with no soil surrounding. Thus it appears that tube volume changes were about the same in both types of tests.

In view of these considerations it appears highly unlikely that the tube conformed to soil strains as large as those observed in hollow soil

cylinder tests.

The second basic condition is that the tube underwent only those strains it would normally experience in elastic deformation under a load of, say, 2.5 psi. These deformations are so small compared to the strains necessary to mobilize the full friction angle of the sand that the condition is essentially one of no volume change on the inside of the soil ring. Although it is not readily apparent exactly how the friction in the sand would be mobilized, this seems to be by far the more likely condition to exist in the soil-tube system.

It is entirely possible that the volume changes observed in the hollow soil cylinder tests were a function of the testing technique. For example, interior volume changes were measured by bringing the interior pressure to some value and keeping it constant while the exterior pressure was increased to failure. This is not at all analogous to a soil ring with a tube inside because the tube, instead of exerting a constant pressure on the sand, exerts an increasing pressure as the sand tries to strain inward. Therefore, it is incorrect to assume that the tubes must have experienced the same volume changes as the interiors of the hollow soil cylinders because the two situations are not at all similar.

The weight of all experimental evidence on soil-tube systems indicates that the tubes are subjected only to elastic strains up to their buckling point. The ambiguity which exists between tube and soil strains may well be the result of imposing an improper set of conditions on the soil-tube tests.

L.7 Conclusions and recommendations

Two very definite conclusions can be drawn from the results of the tests. The first is that increases in tube strength with soil surroundings are a direct result of the tubes being forced into higher modes of buckling. Although no way of predicting the number of modes is readily apparent, this

fact in itself suggests that tube strength could be predicted on a rational and relatively simple basis.

The second conclusion is that the failure of the soil-tube system can be analyzed as if it were a hollow soil cylinder through the simple expediency of substituting the tube strength for the pressure acting on the inside of the soil ring. This conclusion has been made in the face of some ambiguity arising from the consideration of strains developed in the soil and in the tubes. It is felt, however, that enough experimental evidence exists to allow this conclusion to be made, especially since the ambiguity may be the result of the imposition of an improper set of conditions.

It would be highly desirable to support both conclusions with more experimental work. All of this work could be of the same relatively simple type previously done. For example, the first conclusion could be checked by running similar tests on tubes of other lengths. Each tube length has a unique set of buckling strengths, and it would be a simple matter to check to see if these strengths were being realized in tests on longer and shorter tubes.

The second conclusion could best be checked by tests using other soil materials and other soil thicknesses. In short, the conclusion can be made a certainty only by an overwhelming weight of experimental evidence. In the realm of soil-tube tests, not much can be done to clear up the strain question. Although not strictly within the scope of this report, it is suggested that a simple test on hollow soil cylinders could clarify the question.

In this test the interior of the cylinder would be kept at constant volume by adjusting the interior pressure while the exterior pressure was brought up to failure. This could easily be accomplished by connecting the interior of the cylinder to a constant-volume pore pressure measuring device used in triaxial testing. If the magnitude of the pressure increases

(which is necessary to prevent volume changes of the interior) turned out to be comparable to pressures which the tubes could exert, then it would follow that the tubes actually did prevent large strains on the interior of the soil ring. On the other hand, if the necessary pressure increases greatly exceeded those which a tube could exert, a very definite discrepancy would be indicated.

It is felt that the settlement of this question should be the next immediate step in the continuation of soil-structure studies.

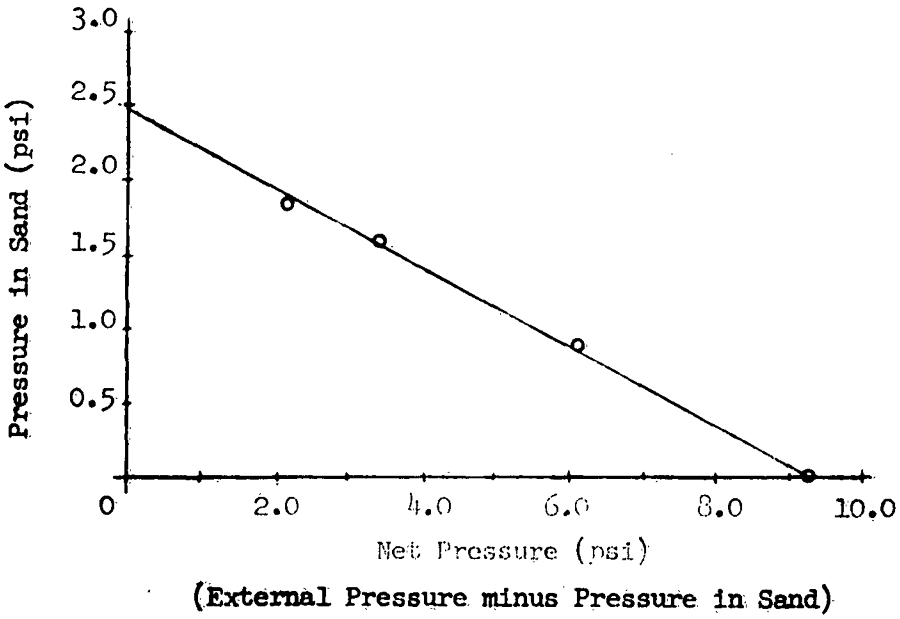

Figure L.1 RESULTS — MODIFIED SOIL - TUBE TESTS

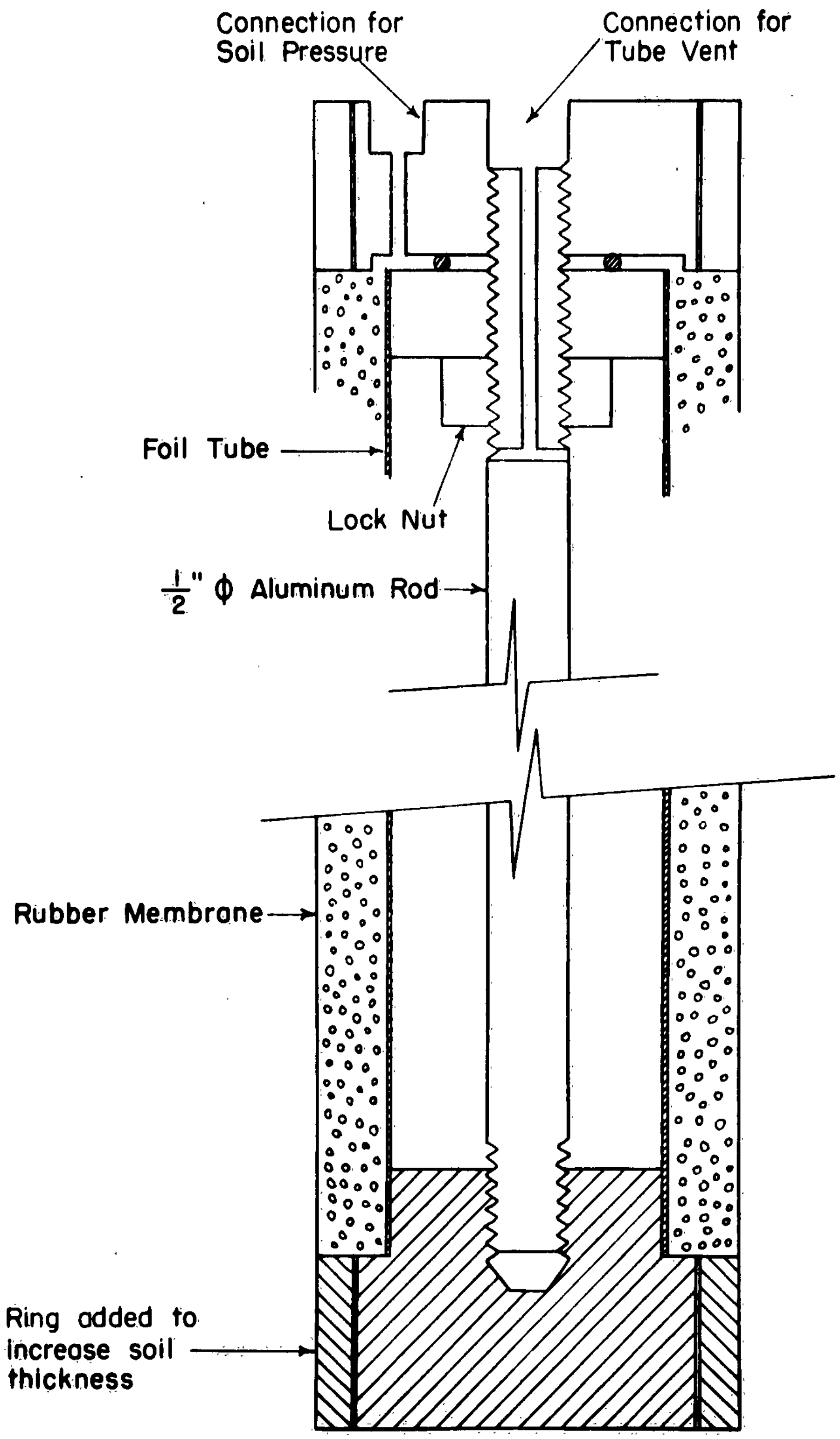

FIGURE L.2 SPECIAL APPARATUS
DRAWN FULL SIZE

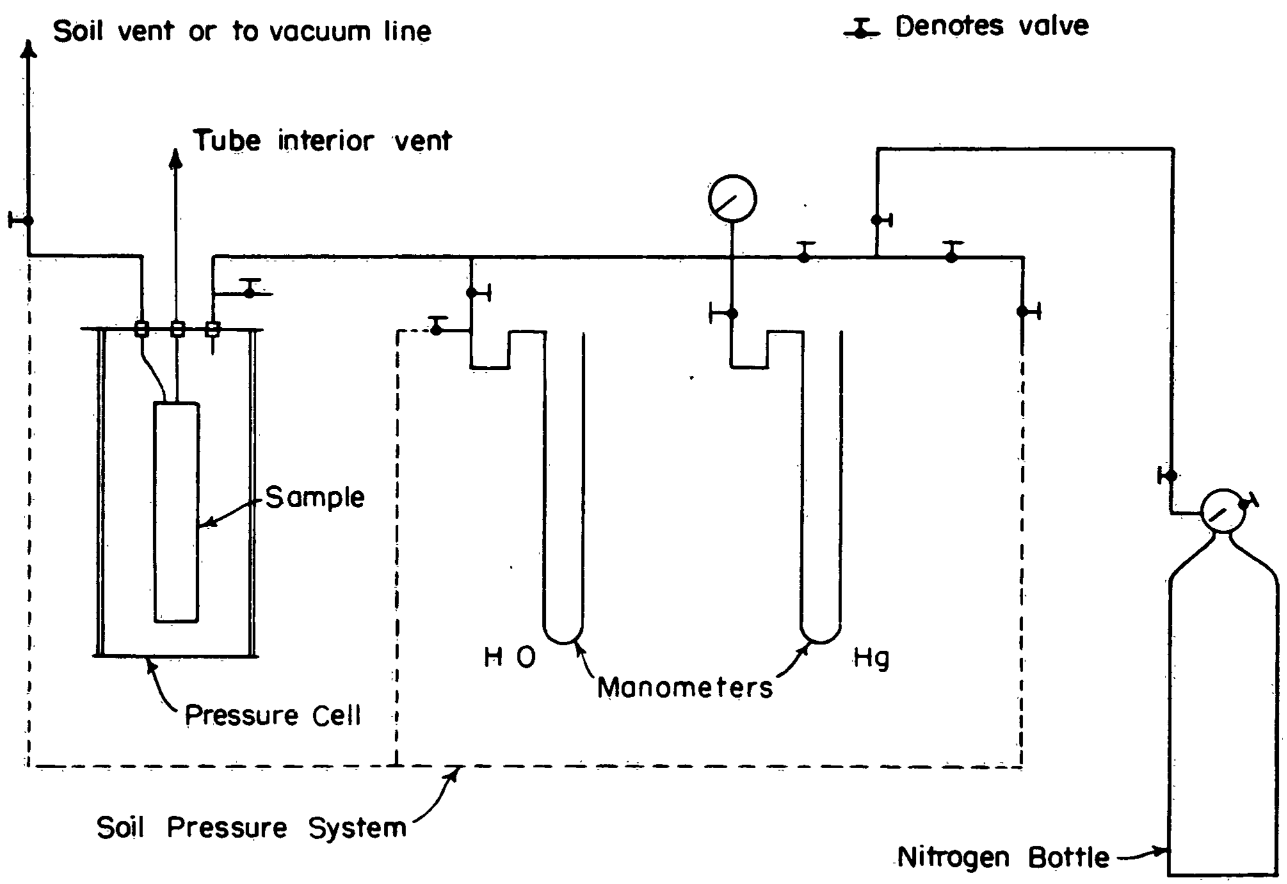

FIGURE L.3 SCHEMATIC OF TEST SET UP

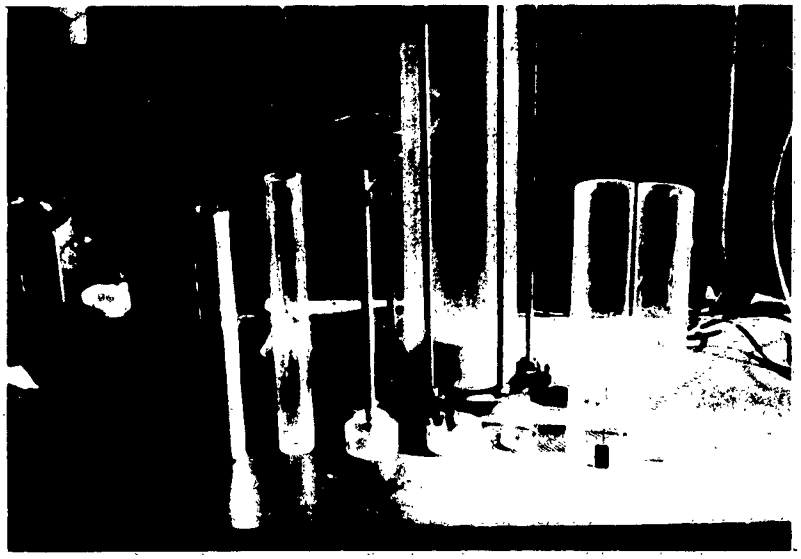

Figure L.4 Apparatus

Pictured from left to right are:

new tube

tube mandrel

base piece with axial rod

two top pieces

split mold and tamping rod

Pressure cell is just to the rear.

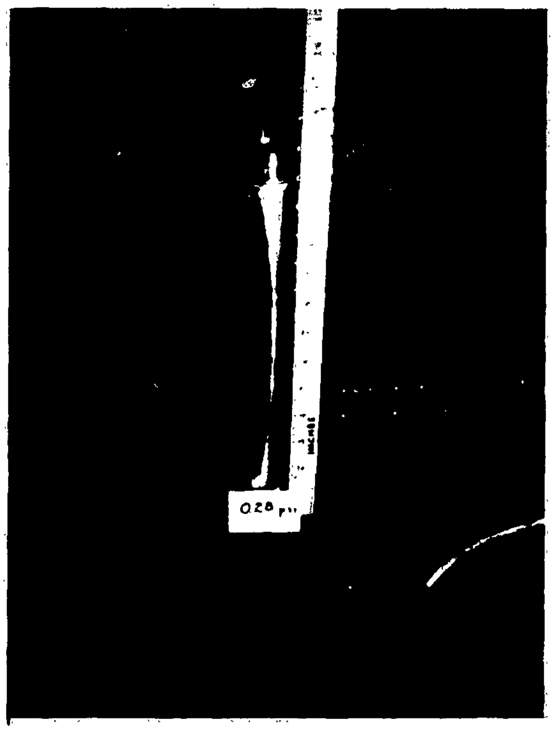

Figure L.5 Tube Buckled without Soil

I. Control Tests

All tubes in these tests buckled with four modes in perfect symmetry. In some cases where the pressure was relieved quickly only two of the four modes actually appeared and the two "sides" developed by buckling were always adjacent. No difference in buckling strength was observed between freshly made and old tubes.

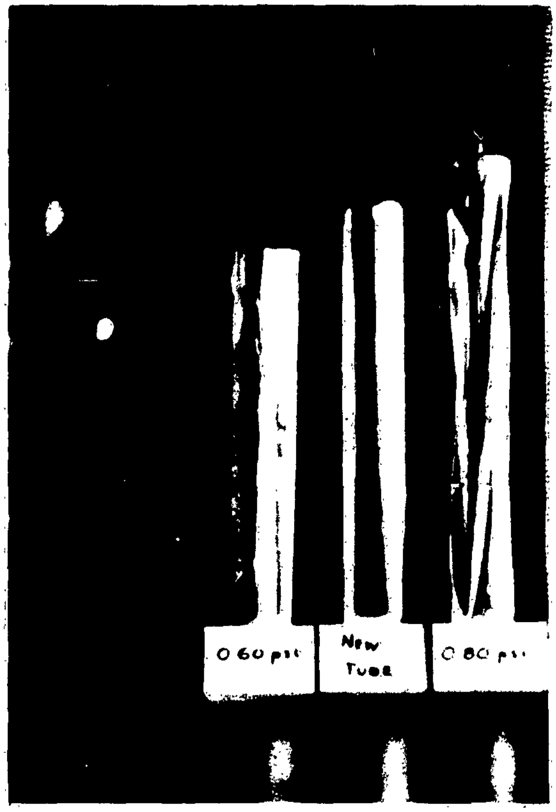

Figure L.6 Tubes Failed in Restrained Buckling Tests

II. Restrained Buckling Tests

Tubes in these tests tended to fail in a manner similar to those failed without restraint. In those cases where the sand was tightly compacted around the tube, the outer surface of the tube was pocked by the sand grains.

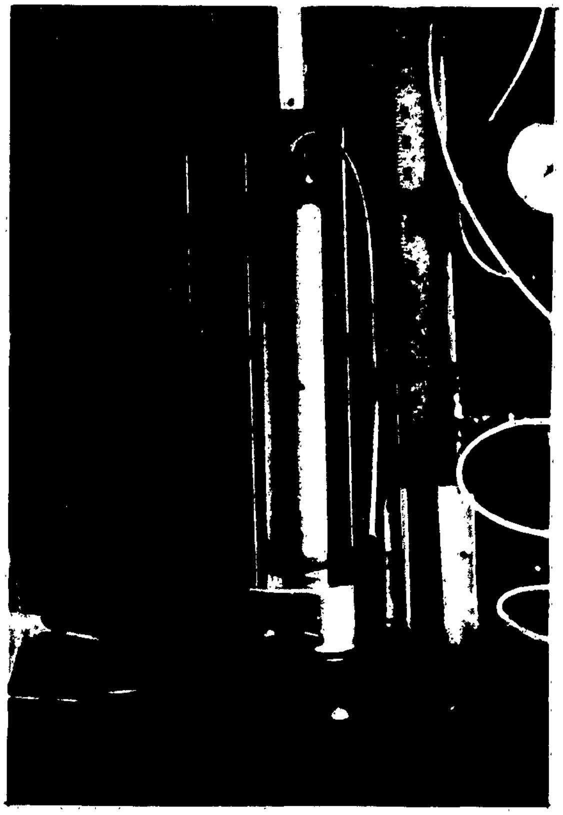

Figure L.7 Soil Surrounded Tube Before Testing
Support is provided by vacuum in sand.

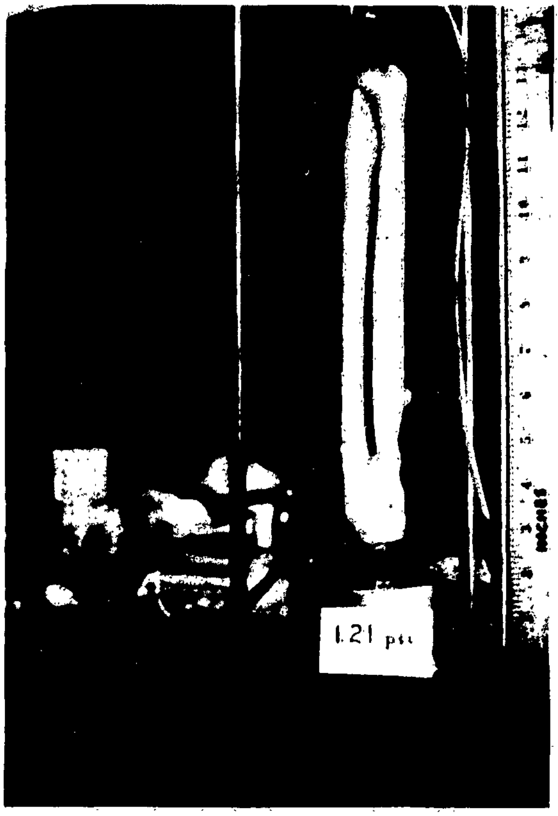

Figure L.8 Failure of 3/16" Test

III. Soil-Tube Tests

Failures in these tests are characterized by sudden and rapid collapse of the soil-tube system. The failure is slightly less dramatic in the 3/16 inch wall samples than in the thicker walls as is illustrated by Figures L.8 and L.9. Failures in all tests were, in general, similar to those pictured.

Figure L.9 Failure of 3/8" Test - **Soil Surrounded Tubes**

The outer surface of tubes in the 3/16 inch wall tests was generally pock marked by the sand grains. In the thicker walls only occasional marks were observed. In all cases the inside walls were completely unmarked.

Figure L.10 Tube After Failure -- 7/16" Test

Collapse of the soil system resulted in complete destruction of the tube (see above), making it impossible to analyze where or how tube failure first occurred.

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2. TIMOSHENKO, S. (1936): Theory of Elastic Stability, McGraw-Hill Book Company, Inc., New York.

Appendix M

SOME EXPERIENCES WITH SIDE-FRICTION

Appendix M
SOME EXPERIENCES WITH SIDE-FRICTION

M.1 Introduction

Considerable research has been done at M.I.T. in the past concerning the magnitude of side friction during consolidation tests, and upon means for reducing this side friction. Since this information bears upon the problem of boundary effects in soil bins, and because the M.I.T. investigations are reported in theses which are not generally available, the results of these experiments are briefly summarized here. All of the quoted results were obtained using Boston clay.

M.2 Side friction in standard consolidation tests

HILTNER (1937) designed a special consolidometer in which a direct measurement of the side friction force could be made. This was achieved by supporting the sample barrel independently from the sample base, through a proving ring, which measured the frictional force transmitted to the sample barrel.* Hiltner obtained data with this device, and the device was subsequently used by BURROW (1948) for additional studies. The device was fabricated from brass and the soil sample pressed directly against the sides of the consolidometer barrel. The consolidometer had an inside diameter of 4.25 inches, and the standard sample height was 1.25 inches.

In a series of tests upon Boston clay, Burrow found that the side friction forced averaged approximately 20% of the vertical load increment. Figure M.1 shows the manner in which this side friction force increased as a function of time. One should keep in mind that the sample underwent consolidation subsequent to application of load increment. During this period of consolidation , the effective stresses within the sample

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* A description and a sketch are also given in M.I.T.(1942).

increased as the excess pore pressures dissipated. Since the magnitude of the side friction should be related to the magnitude of the lateral effective stresses, the side friction force should increase throughout the consolidation interval. The theoretical time for 90% consolidation in a Boston clay sample of this thickness, with double drainage, is approximately 20 to 40 minutes.

M.3 Side friction with various surface treatments

DE WET (1953) used this same special consolidometer to study the magnitude of side friction developed when various surface treatments were used adjacent to the inside surface of the consolidometer barrel.

In the first series of tests, the soil sample was separated from the consolidometer barrel by a thin rubber membrane, and graphite was placed between the rubber membrane and the barrel. A typical side friction versus time curve appears in Figure M.2. It may be seen that the side friction built up to its peak value in only five minutes, after which it increased only slightly until the end of primary compression. There was only a slight additional increase in side friction during a subsequent five day period of secondary compression. In general, the average maximum side friction was 12% of the load increment. Thus, the graphite-lubricated rubber membrane served to reduce the side friction to approximately 60% of that observed in the "standard" test. The membrane also caused a significant change in the time build-up of the side friction force, since now the side friction was related to the total stresses within the soil sample rather than to the effective stresses.

In the second series of tests, the soil sample was again surrounded by a thin rubber membrane coated with graphite, but now the inside of the brass consolidometer barrel was lined with Teflon. A typical friction versus time curve appears in Figure M.3. In these tests, the side friction built up to a maximum value within the first few minutes, and then,

as the primary compression continued, decreased to 2/3 or less of its peak value. For a 2 ton/ft^2 load increment, the maximum side friction amounted to approximately 12% of the load increment (this is the case shown in Figure M.3), whereas with a 4 ton/ft^2 load increment, the peak side friction was only 6% of the load increment.

The final situation investigated by De Wet involved soil bearing directly against the Teflon-lined consolidometer barrel. Once again the magnitude of the side friction should be related to the effective stresses within the soil sample, and, as may be seen in Figure M.4, the side friction built up much more slowly than it did with the rubber membrane. With this surface treatment, the maximum side friction was approximately 10% for a 2 ton/ft^2 load increment and only 3 percent for a 4 ton/ft^2 load increment.

ALDRICH (1951) also made certain studies concerning the effects of using a graphite-lubricated rubber membrane. Aldrich was measuring pore pressure at the mid-plane of a double draining consolidation specimen, and the rubber membrane was necessary for his pore pressure measuring technique. Aldrich did not measure the side friction directly, but rather compared the shapes of compression versus time curves obtained with and without the rubber membrane. His principal finding was that the experimental compression versus time curves agreed with the theoretical curves much better for the tests in which a rubber membrane was used than for the tests not using the membrane. Such a result is to be expected, since the side friction force tends to be constant when a membrane is used but is a variable force if the soil bears directly against the consolidometer barrel.

M.4 Conclusions

These results show, first of all, that the side friction, developed where soil is in contact with an ordinary metal surface, is very large indeed.

The results further show that the magnitude of the side friction can be materially reduced, especially by use of a Teflon coating over the inside of the consolidometer. As a result of these findings, Teflon-coated consolidometers have been used extensively in the M.I.T. laboratory. However, even with the Teflon coating, the friction force in a ring with a diameter/height ratio of 3.4 to 1 has amounted to as much as 10% of the vertical load increment. Such a friction force would become prohibitedly large as the diameter to length ratio is decreased.

De Wet suggested that still further reduction of side friction might be achieved by rubbing the membrane with a Teflon powder in addition to using the Teflon-coated barrel. Sheet Teflon has become available since these tests were conducted, and its use, together with a Teflon coating over the metal barrel, would presumably accomplish the same objective.

Finally, it should be noted that relatively large axial strains developed in these tests upon Boston clay. Thus, there was ample opportunity for full mobilization of any potential friction forces.

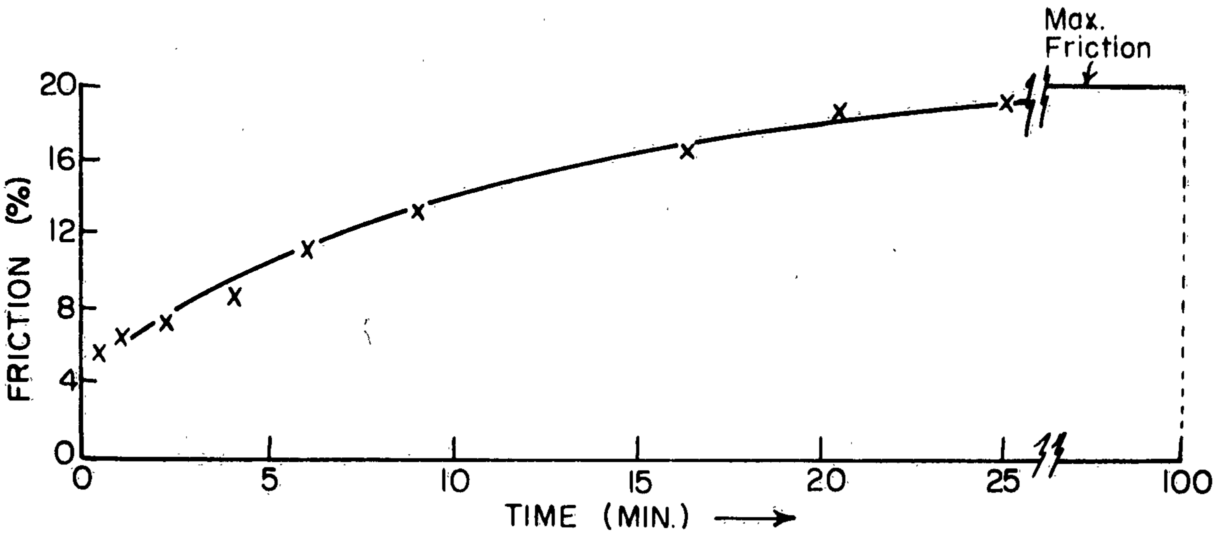

FIGURE M-1

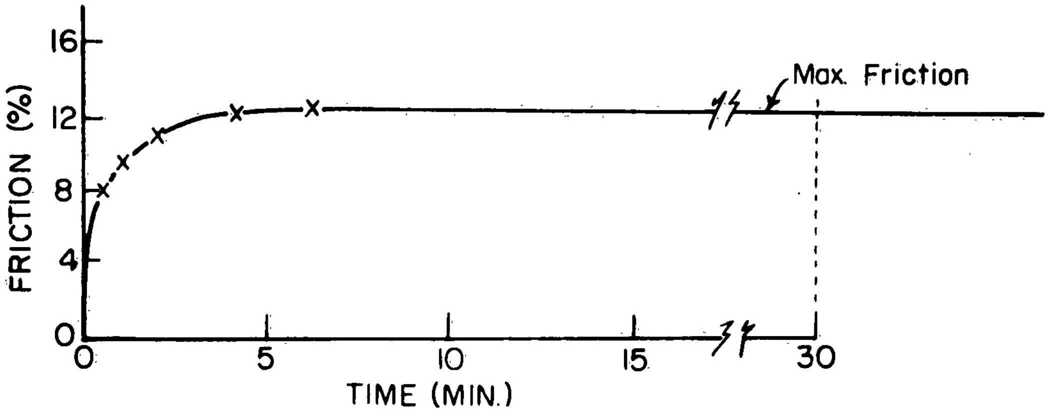

FIGURE M-2

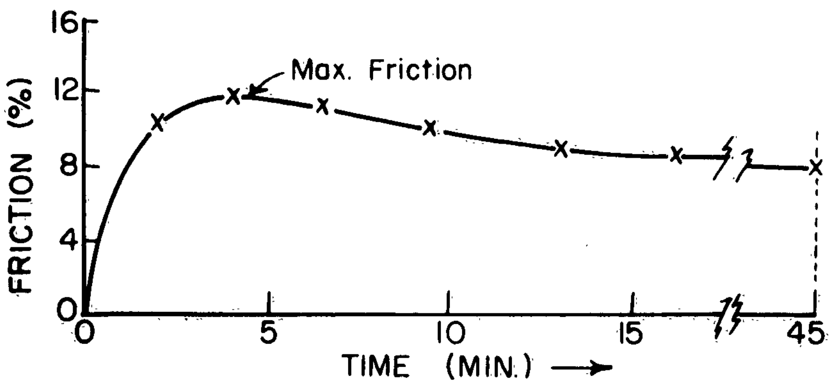

FIGURE M-3

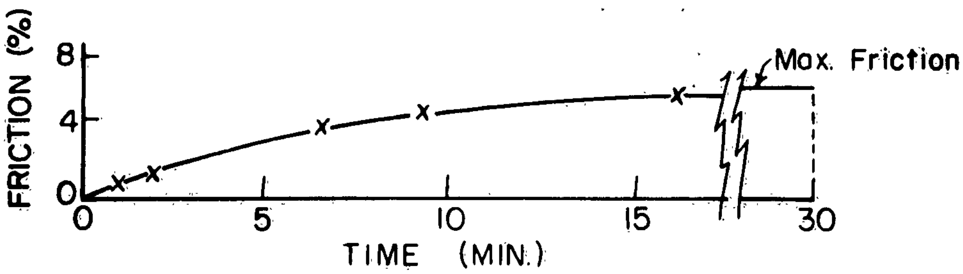

FIGURE M-4

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Appendix N

FRICTION ANGLES OF SANDS AT LOW CONFINING PRESSURES

Appendix N

FRICTION ANGLES OF SANDS AT LOW CONFINING PRESSURES

N.1 Introduction

This appendix describes the work performed to investigate the effect of very low confining pressure on the angle of internal friction of a cohesionless material.

N.2 Background

The friction angle of a cohesionless soil is known to vary with the normal pressure applied to it. In the extreme case of a single sand grain resting on other sand grains cemented to a flat surface, the surface would have to be tilted to an angle much larger than the friction angle of the sand in order to make the single grain rollout of the pocket into which it had fallen.

TAYLOR (1938) observed an increase from 29 $\frac{1}{2}^{\circ}$ at a pressure of 8 tons/ft^2 to 34 $\frac{1}{2}^{\circ}$ at $\frac{1}{2}$ ton/ft^2 in Ottawa Standard Sand. Studies of model footings in sand have indicated that this effect may be much more pronounced at extremely low confining pressures: (see Appendix B). An article appearing in Engineering on 30 May 1930, published results of tests performed on Ottawa Standard Sand at normal pressures as low as 2 psi. Friction angles as high as 50° were reported, and the results were extrapolated to 61° for a pressure of $\frac{1}{4}$ psi. These tests were run in a device similar to a direct shear machine, but the shear boxes moved over each other on rollers and it is quite possible that frictional effects in the apparatus contributed to the high values obtained for the friction angles.

The following tests were undertaken to obtain more information

concerning the exact magnitude of this effect.

N.3 Procedure

N.3.1 Testing equipment

Direct shear tests were selected as a simple method of obtaining the friction angles of a sand at various normal pressures. These tests are usually run in a shear box which has dimensions of 3" x 3". The low confining pressures planned for testing made a box of these dimensions impractical because of the extremely low normal loads involved. For this reason a box of 12" x 12" dimensions was employed.* Using this box, the lowest practical load is about 36 pounds of ¼ psi, and the capacity of the apparatus limits the highest load to 1500 lbs or approximately 10 psi. A limited number of tests were run in the 3" x 3" box as a check on the results obtained in the large box. All tests were run on the MIT direct shear machine, the procedure being as outlined in Soil Testing for Engineers by T.W. Lambe.

N.3.2 Samples

Two types of sands were tested. The first was a graded Ottawa Sand. This is distinguished from Ottawa "Standard" sand in that it is not a uniform material, but consists of both medium and fine sand grain sizes. An initial void ratio of approximately 0.53 (a medium dense condition) was found to be most easily reproduced; however, this value varied between 0.53 and 0.55 for the tests. At least three, and in some cases, five tests were run at normal pressures of ¼, ½, 1, 2½, 5, and 10 psi.

The second material was a uniform coarse angular sand obtained from a gravelly beach sand by removing all but the coarsest sand sized grains. The initial void ratio of this material was set at 0.78, a medium dense

* This box was used by Taylor to study the size effects of shear boxes. He reported a reduction in friction angle of 1½% for Ottawa Standard Sand tested in the 12 inch box.

condition. Control of the initial void ratio was greatly improved and this value was the same for all tests. Tests were run at the same normal pressures as used on the Ottawa Sand with the exception of a set run at 2.2 instead of 2.5 psi.

In addition to the tests in the 12 inch box, both sands were tested in the 3 inch box at a normal pressure of 10 psi. Initial void ratios were the same as used for each material in the large box. As a check on the apparatus two tests were performed using the small box on uniform Ottawa sand (20-30 mesh) for comparison to accepted experimental values.

N.4 Results

N.4.1 Friction angle versus normal pressure

Angles of internal friction versus normal pressures have been plotted in Figure N.1 for both the Ottawa and coarse sand. Only a negligible increase in friction angle was observed at the lowest pressures on the Ottawa Sand. A definite increase in friction angle of about 3° was observed in the coarse material in the range from 10 psi to $\frac{1}{4}$ psi.

Tests performed on the two materials using the 3 inch shear box produced the same results at 10 psi as the tests performed in the 12 inch box. The two tests on Ottawa Standard sand were in good agreement with the values obtained by Taylor for the same material.

N.4.2 Volume change versus normal pressure

The average volume changes at peak stress for various normal pressures are tabulated in Table N.1. The Ottawa Sand showed approximately the same volume increase at all normal pressures, whereas the coarse sand had a volume decrease at the higher pressures, and a volume increase at the lower pressures.

Normal Pressure (psi)	Volume Change (%)	
	Ottawa S.	Coarse S.
0.25	.51	.81
0.50	.56	.66
1.0	.51	.20
2.5	.51	-.24
5.0	.46	-.76
10.0	.55	-.89

% VOLUME CHANGE AT PEAK SHEAR STRESS

TABLE N.1

N.5 Discussion

N.5.1 Effect of variation of initial void ratio on friction angle - Ottawa Sand

The results of the tests on Ottawa Sand show considerable variation in friction angle for tests run at the same normal pressures. This error is the result of failing to obtain the same initial void ratio for each test. Measurements indicate a possible variation of 0.53 to 0.55 for the initial void ratio value, or an error of about 4%. Based on results of tests performed on Ottawa Standard sand, a variation of 4% in void ratio in the dense condition will change the friction angle by $1\frac{1}{2}^{\circ}$. The largest variation observed in the tests was $1\frac{1}{2}^{\circ}$ at 1 psi. Even considering the extreme values observed at each normal pressure, only a small variation in friction angle is present between 10 and $\frac{1}{4}$ psi.

N.5.2 Friction angle versus normal pressure - Coarse Sand

The tests on the coarse sand were undertaken with the expectation that the variation of friction angle would be more pronounced for that type of material. The results show no variation of friction angles at the same normal pressures as was experienced with the Ottawa Sand. This consistency may be attributed to either better control of the initial void ratio or the fact that the initial void ratios had less effect on the friction angles because sand in a less dense condition was used.

A definite increase in the friction angle was observed as the normal pressure decreased. The $27\frac{1}{2}^{\circ}$ observed at 10 psi increased, with each successive decrease in normal pressure, to a maximum of about $30\ 2/3^{\circ}$ at $\frac{1}{4}$ psi. Although these results clearly indicate a trend, the variation of 3° in this region is relatively small, the actual increase in coefficient of friction (or tangent of the friction angle) being only from 0.51 to 0.60.

N.5.3 Validity of experimental results

Examination of the results of the tests on the two sands seemed to indicate that the values of friction angles obtained were too low. A careful check of calibrations and measured values failed to reveal any source of systematic error, so it was decided that the large shear box could possibly have contributed to the low values. The tests performed in the smaller box eliminated this possibility. As a final check for possible errors two tests were run on Ottawa Standard sand. The measured friction angles were consistent with accepted values. All checks, then, point to the validity of the results obtained.

N.5.4 Relationship of change in friction angle to volume change

Table N.1 showing volume changes during shear has been prepared to demonstrate the relationship of change in friction angle to volume change. (An increase in friction angle may be predicted for an increase in volume change since the work done while producing the volume increase must be provided by an increase in shearing force). The experimental results indicate that such is the case.

Volume increases during shear are caused by the physical interference of the individual sand grains with the result that particle displacements can be accomplished only by forcing the grains to climb over each other. The amount of volume change (and hence the additional amount of shearing force) resulting from this interference appears to be related to the type of material, and to the normal pressure involved.

The Ottawa Sand, which is fairly well graded and is composed entirely of rounded particles of quartz, exhibited essentially the same volume change during all tests. The poorly graded coarse material is composed of angular particles of quartz and feldspar and had entirely different volume changes at different normal pressures. At conventional testing pressures (10 psi) the coarse sand behaved like a loose material, but at $\frac{1}{4}$ psi its volume change during shear was similar to that of a dense

material.

N.5.5 Role of shear displacement in the friction angle - volume change relationship

It might be postulated that for a given material and a given normal pressure, there exists a unique density on the failure surface at which shear strains can occur. A portion of the shear strength of a material is represented by the work done to bring the particles in the failure zone to this density.

Continuing this hypothesis, the change of this density at different normal pressures may be explained by the existence of two separate mechanisms of particle displacements relative to each other. The first, occurring at high normal pressures, consists of a sliding of the particles over each other, with little or no change in orientation. As the normal pressure is decreased, there develops a tendency for the particles to rotate as they are displaced. Unless the particles are perfectly round, any rotation results in increased volume changes. The actual mechanism of shear displacement is undoubtedly a combination of both sliding and rotating movements. The degree to which one or the other predominates, and the corresponding volume change, would then be a function of normal pressure.*

Gradation of a material is another important consideration. A well-graded material can undergo particle displacements with relatively small variations in density be a rearrangement of the small particles occupying the voids between the larger ones. A uniform material, however, lacks this flexibility of structure, and particle movements are likely to produce

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* This hypothesis was suggested by A.R. Philippe. Another explanation is that grains pass "over the mountain tops" at low confining pressures and "through the passes" at high confining pressures. In any event, the sample develops shear strains by the easiest course open to it, and if expansion makes shearing easier, the sample will tend to expand. See TAYLOR (1948) for the standard explanation of the effects of volume expansion on friction angle.

a more drastic change in the density of the material.

Considering the experimental results, the hypothesis seems reasonable. The Ottawa Sand, which approaches being perfectly round and is well-graded, showed little variation in volume change and correspondingly little difference in friction angle. The poorly graded coarse material exhibited increasing volume changes with decreasing normal pressures along with increasing angles of internal friction.

N.6 <u>Conclusions</u>

Increases in friction angles upon decreasing normal pressures are associated with the additional volume changes which the material undergoes during shear. The magnitude of these changes is dependent on the properties of the material and on the normal pressure during the test. On the basis of tests on only two materials, it is not possible to accurately predict variation in friction angles for any material, but it is possible to define the extremes. Little or no variation can be expected in well-graded, round materials, and maximum variation of friction angle can be expected in uniform angular materials.

The increase, however, is evidently not spectacular at very low pressures. The best indication is that the friction angle is a linear function of the log of normal pressure, i.e., the increase in friction angle from 10 to 1 psi is the same as the increase from 1 to 0.1 psi. For the two materials tested, increases were about $1\frac{1}{2}^{\circ}$ per cycle for the coarse material, and $1/3^{\circ}$ per cycle for the Ottawa Sand.

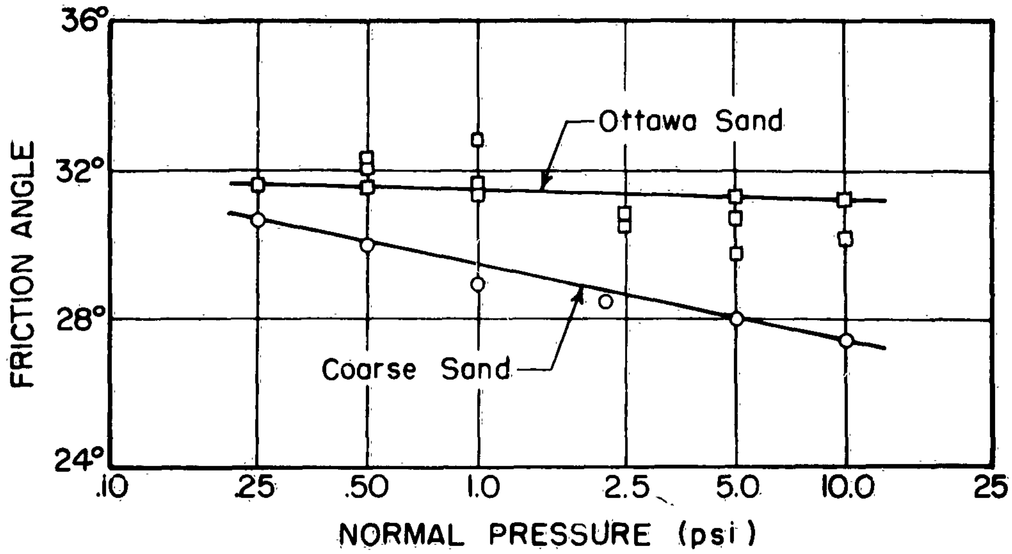

FIGURE N.1 FRICTION ANGLE VS. NORMAL PRESSURE
Ottawa Sand and Coarse Sand

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